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SHIP HULLS MADE OF REINFORCED CONCRETE
(Korpusa sudov iz amotsementa)

(Design, Strength, and Construction Technology)

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239

TABLE OF CONTENTS

Page

CONVENTIONAL SYMBOLS

R_p = standard tensile strength of reinforced concrete

R_c = standard compressive strength of reinforced concrete (prismatic strength of concrete)

$R_{p.и.}; R_{c.и.}$ = standard tensile and compressive strength of reinforced concrete during flexure

R_{ck} = standard shear strength of reinforced concrete

R_{cp} = standard cutoff strength of reinforced concrete

R_a = design strength of extended rod framework (reinforcement) used for reinforcing the concrete

E_p = modulus of elasticity of reinforced concrete under axial elongation

E_c = modulus of elasticity of reinforced concrete under axial compression

$E_{p.и.}; E_{c.и.}$ = reinforced concrete's modulus of elasticity for expansion and contraction during flexure

G = modulus of shear for reinforced concrete

γ = coefficient of relative transverse deformation of reinforced concrete (Poisson coefficient)

$\gamma_{ац}$ = volumetric weight of reinforced concrete

$\gamma_{п.б}$ = volumetric weight of cement-sandy concrete

$K_п$ = specific surface of reinforcement (total surface of wire in grids per unit of volume of reinforced concrete element)

μ = coefficient of reinforcement (ratio of area of cross section of longitudinal reinforcement to cross sectional area of element)

l = actual length of design's element

l_o = computed length of design element under calculation for stability

b = least dimension of rectangular cross section of element

r = minimum radius of inertia of element's cross section

INTRODUCTION

Among the design hull materials in use for shipbuilding, a specific place is occupied by ferro-concrete and one of its variants, i.e. reinforced concrete.

The substantial savings in metal (by two-three times) realised the construction of the hulls of floating structures of reinforced concrete, at a simultaneous substitution of expensive and scarce sheet and profile rolled iron, with reinforced rod steel, establishes that unvarying interest which is expressed in ferro-concrete in shipbuilding. Just as much interest in the floating facilities with ferro-concrete hulls is also manifested by the operating organizations, since in distinction from the structures with steel hulls, the designs with ferro-concrete hulls do not require layovers in dock for painting the hulls and for the periodic replacement of rusted plates of sheathing and elements of a set, which saves considerable resources. In spite of the circumstances noted above, confirmed by the construction experience and the subsequent operation of the ferro-concrete floating facilities, as a whole for the ferro-concrete shipbuilding for the past 50 years, we have typically had both periods of abrupt increases, and periods of equally abrupt declines, including the complete shutdown of this type of floating facilities.

Such an instability in the development of the ferro-concrete shipbuilding, in addition to the purely subjective factors (and in a number of cases, of outright prejudice), has been established mainly by the absence of the necessary objective conditions for the regular construction of ferro-concrete hullc reliable in operation.

The seemingly simple organization of construction, the simplicity of the production processes under the possible utilization of specialists with relatively

low skills have inevitably involved the appearance of superfluously overheavy, over-reinforced hulls with a low quality of concrete, cracking under the effect of atmospheric moisture, fluctuations in temperature and in sea water. A time was required sufficient in r that the basic conditions, determining the reliable operation of the marine ferro-concrete hulls, were revealed, understood and strictly regulated. Beginning from 1955, as a result of the accomplishment of a series of scientific-research, planning and testing-design activities, the necessary scientific base was created for developing the plan of ferro-concrete ships and for the selection of the most effective engineering processes for their building.

The systematic studies which were conducted established the necessary extents of reinforcing the main and secondary design elements, the principles of efficient design configuration; standard documents were developed for all types of work in the assembly and concrete reinforcing both of the individual sections, and of the entire hull as a whole; the specific requirements were determined and delineated on monitoring the qualities of the finished products. As a result of the research activities, it was demonstrated that the conditions of working on concrete reinforced hulls, situated in sea water under constantly changing loads (in amount and sign), the requirements of resistance to freezing, complete airtightness and reliable resistance to the aggressive effect of sea water basically distinguishes the floating river and naval ferro-concrete facilities from any civil installations, including the hydrotechnical ones.

The book brought to the attention of the readers contained a brief discussion of the basic results of the activities and studies conducted by the shipbuilding scientific-research organizations, the design bureaus and the shipyards engaged in reinforced concrete shipbuilding, with the development of design criteria and engineering processes necessary [illegible] planning and building the hulls of floating facilities made of reinforced concrete.

The book also presents data from published reports (on reinforced concrete) issued by the leading specialized organizations: the NIIZHB of the USSR Gosstroy, the Len ZNIIEP, the NIISK Gosstroy of the USSR, the TNIISGEI, the ISiA of the Byelorussian SSR Academy of Sciences and the NIIsel'stroy.

The technical advantage of using reinforced concrete in the designs of the hulls and superstructures of ships has been confirmed by the experience gained in the construction and operation of various reinforced-concrete ships in our country and abroad.

In the building of the first reinforced-concrete ships, initially there was a definite uncertainty in the new shipbuilding material, especially in respect to the capability of the elements to function during impact and sign-changing loads.

In spring of 1943, in Italy extensive research and experiments were conducted under the supervision of the Naval administration. The purpose of the experiments was the establishment of the actual values of the physico-mechanical characteristics of reinforced concrete under the effect of permanent and impact (shock) loads. The experimental plates with a dimension of 1.5 X 1.5 m, with a thickness of 30 mm, reinforced by thin wires with a total expenditure of metal amounting to 400-500 kg per cubic meter of concrete, were tested by dropping a load on them weighing 260 kg from a height up to 3 meters. At this time, it was established that even with the presence of small and partial cracks, in the region of strain, the plates retained their water tightness.

The construction (begun in 1943 by the Nervi and Bar oli Firm) of three motor-driven transport ships for the navy and a motor-driven transport ship with a cargo capacity of 400 tons was interrupted owing to the war. The construction was renewed in 1946 at the Lazzarini and Meacn. Shipyard in Anzio. The motor-driven speed sailboat "Irene" with a capacity of 165 tons was built

in three months (Fig. 1). The thickness of the hull sheathing on the "Irene" equalled 35 mm. The plating was made of eight layers of reinforced screens with a grid size of 1 cm and a weight of 1 kg/m^2. Four layers of the screens were located closer to the external, and four layers closer to the inner surfaces of the plating. Between the layers of screens, three rows of steel reinforcing rods with a diameter of 6 mm were placed, arranged 10 cm from each other. The reinforcement grids and rods were tightly interconnected by a steel wire; in the preparation of the cement-sandy solution, for the plating, 1000 kg of cement per cubic meter of concrete were used. The placement of the cement-sandy solution on the grids was accomplished manually (by workmen) from the inside of the hull. The shaping of the hull was accomplished without the application of cement forms.

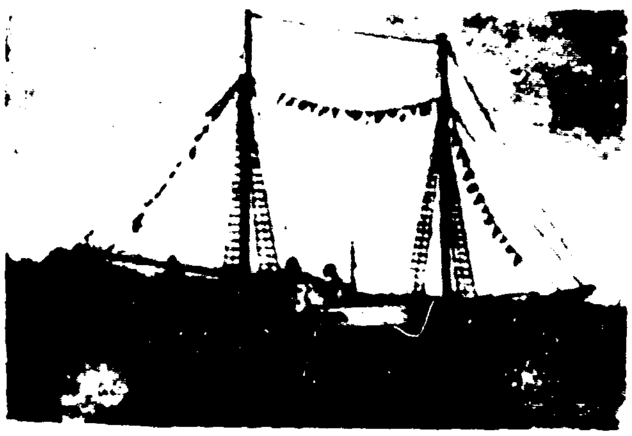


Fig. 1. Motor-Driven Reinforced Concrete Sailboat "Irena".

The experience gained in operating these ships removed all doubt in respect to reinforced concrete as a shipbuilding hull material. The ships demonstrated high operational-technical qualities. After 10-20 years of operation, the hulls were in good condition, without showing any appreciable damages. The motor sailboat "Irene" during the operating period from 1945 to 1958, making regular cruises under stormy conditions between various Italian ports, did not have any significant damages. The occasional damages to the hull from collisions

during docking were only local blind cracks without loss of the water tightness of the plating. Such damages were repaired by the crew without putting the ship out of operation.

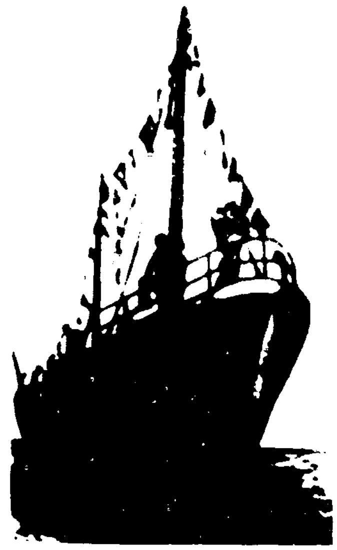


Fig. 2. The Reinforced-Concrete Fishing Trawler "Santa Rita".

Other reinforced-concrete ships built in Italy from 1945-1948 (the excursion yacht "Nennele", having a hull plating thickness of 10-12 mm, reinforced by seven layers of gratings with a mesh size of 1 cm and a weight of 1 kg/m^2, the fishing motor-driven ship "Santa Rita" with a deadweight of 165 tons, (Fig. 2), is still in operation.

In hull weight, the concrete-reinforced ships built in Italy are 5-10% lighter than the wooden hulls of similar ships, while their cost proved to be less by 40-60%.

The reinforced-concrete navigational launches (Fig. 3) built by the British firm Windboats are equal in their operational-technical indexes to the same launches with a hull made of wood or steel. They have a lighter hull, equally as strong as a steel hull. The expenditures for the construction of the reinforced-concrete launches prove to be substantially lower (by 55-80%) than the costs for building the launches with a hull made of steel, wood and

fiberglas. The low operating costs and the simplicity of repairing the reinforced-concrete hulls increase still more their economic effectiveness.

The repair of occasional accidental damages to the hull of the cutters were as a rule conducted by the crews. Thus, one of the cutters, as a result of a collision of a yacht in its stem, sustained damage in the amidships region with a dimension of 0.75 X 0.60 m. At this time, the maximum sagging of the reinforced concrete plating reached 4 cm. After the elimination of the sagging with the aid of a hydraulic jack, in the plating we found only minor cracks with a depth up to 3 mm. The damage was repaired with cement-sandy concrete in 30 minutes.

Fig. 3. The Reinforced-Concrete Sport Launch Built by the Windboats Firm.

Equally as high operating qualities were also shown by the reinforced-concrete ships built in our country. On 1 July 1957, we launched on the Volga River the first reinforced-concrete yacht "Opyt" (Experience). As is described by engineer I. Ya. Glan, "The first reinforced-concrete yacht 'Opyt' in late autumn 1957 was torn from anchor during a severe storm and was thrown onto the rocks on the opposite shore. We were unable to remove the yacht because of the ice jam which had started. The entire autumn, the hull of the yacht was on the rocks, and during the winter it froze into the ice. In the spring, at first glance the hull of the yacht had a sad appearance. The sides were crumpled, but nevertheless the reinforced gratings proved to be undamaged. All that was

required was the work of four men, a bag of cement and several buckets of river sand in order for the yacht hull to be repaired in one day".*

The yachts "Tsementai", "Progress", "Mechta" and the launch "Energostroitel" also proved themselves well during operation.

For three navigational seasons, the yacht "Tsemental" cruised under the most diverse navigating conditions for more than 2,500 miles along the Dnieper River and the Black Sea. The hull of the yacht proved to be quite strong and waterproof.

On the yacht "Mechta" during the navigational season of 1965, the long trip was made along the Volga from the port of Togliatti to the port of Kazan. During the trip, mostly during a wind of force 7-8 points, the yacht showed high navigating qualities and the absence of any damages to the hull.

Fig. 4. Hauling the Finished Reinforced-Concrete Hull of the Yacht "Progress".

In weight of hull, the reinforced-concrete yachts are not inferior to the wooden ones, while in cost of construction they are cheaper by 5 times. Specifically, the cost of building the hull of the yacht "Progress" (Fig. 4) was around 900 rubles, and in the materials for the hull, around 200 rubles were spent. The weight of the hull of reinforced-concrete yachts comprises 40-45% of their displacement, or 50-55 kg per unit of cubic modulus of the LBH. The ballast weight for yachts made of reinforced concrete comprises 30-35% of the displacement.

*I. Ya. Glan. Flying Rock, "Inventor and Efficiency Expert", No. 7, 1962.

The data presented show that the weight characteristics of the concrete-reinforced yachts are somewhat lower than in the same type of wooden yachts.

As the calculations indicate, the application of reinforced concrete for building yachts proves to be quite justified, if they are longer than 8 meters. At a yacht length of less than 8 m, the hull made of reinforced concrete proves to be heavier than the wooden one. With an increase in the dimensions, the weight of the hull of a reinforced-concrete yacht in relation to the wooden one decreases, and at a yacht length of 15-20 m, it comprises a value of the order of 15-20%.

In 1964, in our country we built a self-propelled driftwood hoisting crane equipped with a reinforced-concrete hull and a superstructure (Fig. 5). During the planning of the crane, the form of the configuration and the main measurements of the hull were not changed as compared with the floating crane having a metal hull, which was reflected on the technical-operating qualities of the ship. Thus, the draft of the reinforced-concrete ship proved to be greater than in the metal prototype. Naturally, this could have been avoided by having increased slightly the principal measurements of the reinforced-concrete hull as compared with the steel hull. With an increase in the draft of the reinforced concrete hull, its resistance to movement increased, whereas the main engines and the propulsion unit were taken to be the same as in the metal prototype. The latter circumstance led to a deterioration in the controllability of the floating crane and to a slight reduction in the travel speed. However, in spite of the errors permitted during the planning of the ship, the economic advantage of using reinforced concrete in place of steel as a material for the ship hull was confirmed. In this connection, the consumption of steel was decreased by more than twice, and the cost of building the hull was reduced by 10% as compared with building the metal hulls.

Table 1

Indexes of the Hull Weight and Use of Steel for the Metal Barges, Barges of Standard, Ferro-Concrete, Prestressed "Keramzit"-Concrete and Reinforced Concrete

Type of vessel and cargo capacity	Material in hull	Total volume of ferroconcrete in hull, m^3	Total weight of reinforcement or sheet and profile steel for hull, tons	Weight of steel per ton of cargo capacity, T/T	Weight of hull per ton of cargo capacity, T/T	Remarks
marine dry-cargo barge with hoisting capacity of 300 tons	standard ferro-concrete	70.5	34.0	0.11	0.58	one barge was built from 1943-1944
marine dry-cargo barge with hoisting capacity of 500 tons	the same	120.0	46.5	0.09	0.65	more than 25 barges were built from 1948-1955
marine dry-cargo barge with hoisting capacity of 400 tons	steel	----	129.0	0.32	0.32	based on data from B.V. Bogdanov. Sea & Harbor barges, Sudpromgiz, 1963
seagoing liquid cargo barge w/ hoisting capacity of 1,000 tons	standard ferro-concrete	230.0	100.0	0.10	0.60	technical project
seagoing dry-cargo barge with lifting capacity of 1300 tons	the same	298.0	138.0	0.11	0.60	the same
seagoing dry-cargo barge with lifting capacity of 1000 tons	steel	----	320.0	0.32	0.30	based on data from B.V. Bogdanov. Seagoing & harbor barges, Sudpromgiz, 1963

Type of vessel and cargo capacity	Material in hull	Total volume of ferroconcrete in hull, m^3	Total weight of reinforcement or sheath of profile steel for hull, tons	Weight of steel per ton of cargo capacity, t/t	Weight of hull per ton of cargo capacity, t/t	Remarks
harbor liquid cargo barge with lifting capacity of 1400 tons	standard ferro-concrete	272.0	136.0	0.10	0.49	two barges were built from 1945-1947
barge-platform with lifting capacity of 600 tons	reinforced concrete for plates of sheathing of bottom, side & longitudinal bulkheads; the remaining parts are made of keramzit-ferroconcrete	91.3	38.0	0.06	0.40	predesign processing
barge-platform with lifting capacity of 600 tons	prestressed keramzit-concrete	116.3	28.0	0.05	0.40	one barge was built in 1962
reinforced concrete barge w/ lifting capacity of 1000 tons (Czechoslovakian Socialist Republic)	reinforced concrete	102.0	51.1	0.05	0.27	was built in 1965 in Czechoslovakia

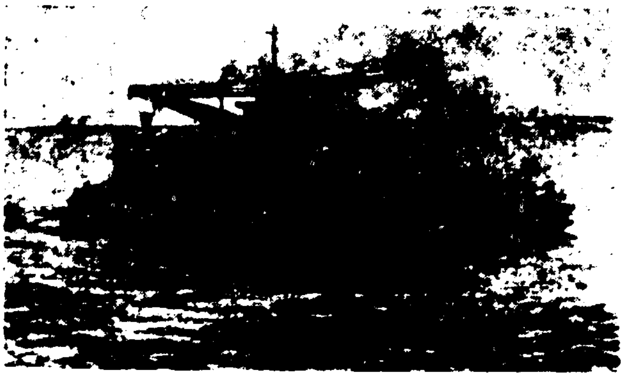


Fig. 5. Reinforced-Concrete Floating Crane with a Hoisting Capacity of 10 Tons.

At the present time, the crane is operating without any restrictions in the lower reaches of the Volga River, after two years of operation, its hull is still in excellent condition all of the high quality elements made during the construction are watertight and do not have any signs of corrosion.

The significant decrease in the weight of hull permits us to utilize effectively the reinforced concrete for the construction of certain types of transport ships, for example barges.

In 1965, in Czechoslovakia, a double-hull reinforced-concrete barge with a lifting capacity of 1,000 tons was built. The weight of the reinforcing steel (gratings and rods) used in building the barge amounted to 51 tons. The weight of the hull in the concrete-reinforced barge referred to one ton of lifting capacity, equals 0.273. The value of the indicated coefficient proves to be close to its value for the metal barges similar in lifting capacity(Table 1). The consumption of metal for building the hull of the reinforced-concrete barge proved to be three times less than in building the hull of a similar metal barge.

The accumulated experience in building and operating the reinforced-concrete ships in our country and abroad permits us to establish that reinforced-concrete can be applied effectively as a hull material for the transport,

fishery, sporting and other ships with a displacement up to 1,000 tons. The reinforced-concrete is also suitable for broad application in the hulls of ferro-concrete ships as a material for making the 'tweendecks', superstructures, deck-houses, etc.

As a result of its resistance to fire, the increased sound insulating capacity and the low heat conductivity, reinforced concrete can be also used successfully in the steel hull as a material for bulkheads, partitions, platforms and foundations.

CHAPTER 1. PROPERTIES OF REINFORCED CONCRETE AS A SHIPBUILDING MATERIAL.

Section 1. Structure of Reinforced Concrete

As a variant of ferroconcrete, reinforced concrete differs from it in its structure.

Ordinary ferroconcrete consists of concrete, reinforced by individual rods or by reinforcing grids, as a rule located in the action sone of the tensile forces. In this connection, reinforcement rods, the diameter of which amounts to 1/8 - 1/10 of the thickness of the ferroconcrete element of a ship design, increases the heterogeneity inherent to concrete. To obtain reinforced concrete, we utilise cement-sandy concrete and thin metal fine-meshed wire, uniformly placed along the section of the design element.

The ship ferroconcrete designs are reinforced with rods having a diameter of not less than 6 mm. The rods or grids with a smaller diameter of reinforcement in the ship design of standard ferroconcrete can not be used, since they do not meet the strength requirements under the observance of the conditions of the technological development (a specific distance between the reinforcement rods, the required stiffness of the reinforcing frames).

The metal grids which are used for the reinforced concrete have a diameter of rods equalling 0,7-1,2 mm, which comprises 1/30-1/40 of the thickness of the element of the vessel reinforced concrete design. The mesh sizes of such grids equal 5-12 mm, whereas the mesh size of the rod gratings which are utilized for reinforcing the ship designs made of standard ferroconcrete comprise 50-100 mm.

By a uniform arrangement of the thin metal grids along the section of the design element in the reinforced concrete, we achieve a distribution of the reinforcement (in distinction from the concentrated reinforcement which occurs in the designs made of conventional ferroconcrete).

From the condition of providing the uniformity of the structure of reinforced concrete through the entire height of the section, the protective layer of concrete in the reinforced-concrete elements comprises 2-3 mm, whereas for the vessel designs, the depth of this layer reaches 10-15 mm.

The dispersed state of the reinforcement provides a much higher specific surface (i.e. related to the volume of material), than in the standard ferroconcrete, of the adhesion of the reinforcement with the concrete, owing to which conditions are developed under which the capacity of concrete to stretch is realized to a greater extent than in the case of the concentrated reinforcement.

The cement-sandy concrete for the reinforced cement is prepared in inert fractions with the exclusion of those coarser than 2.5 mm, whereas for the ferroconcrete, we use mainly the concrete on a base of coarse fillers with fractions of 5-20 mm. In this manner, owing to the application of cement-sandy concretes, in the reinforced concrete, we achieve a structural design which is more uniform through the element's sections.

The specifics of the structure of the reinforced concrete permits us to make more thin-walled designs from it. Thus, while in the concentrated reinforcement according to the conditions of arranging the metal in the ferro-

concrete element, the thickness of the latter in effect could not be less than 4.5 cm, the minimal thickness of the reinforced concrete designs can reach 1-1.5 cm. The maximal thickness of the reinforced-concrete elements (3-3.5 cm) is restricted by the technological possibilities of the qualitative placing of the cement-sandy mixture in the reinforcement frame. However, if required by the conditions of strength, the thickness of the reinforced concrete elements can be increased by introducing an intermediate reinforcing grid or rod (with a diameter of 5 mm), located in the center of the height of the element's section. The introduction of the reinforcement rod is also used for increasing the technological effectiveness of producing the reinforced-concrete designs. With consideration of what has been pointed out, the maximal thickness of the reinforced concrete elements can reach 4.5-5 cm.

The combination of such structural factors as the degree of dispersion of the reinforcement and the utilization of cement-sandy concretes more uniform in their nature, leads in final analysis to higher deformative properties of the reinforced concrete, and also to a decrease in the actual weight of the designs made of reinforced concrete as compared with the equally strong designs made of conventional ferroconcrete.

Section 2. Component Materials of Shipbuilding Reinforced Concrete

The reinforced concrete consists of sandy concrete and of reinforced metal grids; both of these components exert a definite influence upon the physico-mechanical properties of reinforced concrete. In this connection, the elastic-strength qualities of reinforced concrete is the function of both parts; however, the physical properties depend mainly on the qualities of the sandy concrete.

Cement-Sandy Concrete. The quality of sandy concrete determines such important qualities for the shipbuilding reinforced concrete as strength

during compression, watertightness, corrosion resistance and resistance to freezing. In their turn, the properties of standard concrete are determined by the type, the activity and consumption of cement, by the water-cement ratio, by the grain size of sand and by the composition of concrete, by the methods of thickening the concrete mixture, by the periods and conditions of its hardening. It was established that the resistance of concrete to compression and tension depends mainly on the activity of the cement and the water-cement ratio.

In the water-cement ratio, we have an increase in the density and strength of the concrete and hence in its watertightness and resistance to freezing. However, in the case of low water-cement ratios under the usual conditions of production, the cement-sandy mixture does not succeed in becoming placed compactly in the design, which leads to a reduction in the strength and life of the design.

For the production of reinforced concrete meeting the requirements of shipbuilding in respect to strength, watertightness and long life the sandy concrete is made from Portland-cement brands (grades) not below 500, of the following forms: standard, plasticized, and sulfate-resistant.

The brand of cement-sandy concrete for producing the marine reinforced-concrete designs should be accepted not below 400*. For obtaining sandy concretes of grades 400 and 500, the consumption of brand 500 cement amounts to 650-800 kg per cubic meter of concrete with a water-cement ratio ranging from 0.32-0.40.

For increasing the watertightness and the frost resistance of concrete, it is necessary to strive toward a reduction in the water-cement ratio (with allowance for the conditions of a method for pouring the concrete mixture into

* The grade of concrete is accepted tentatively and is characterized by the limit of resistance (kg/cm^2) to compression of a concrete block with an edge of 7 cm, made from concrete of working composition and tested for a period of 28 days in conformity with the standard ON9-373-62.

the reinforced-concrete designs).

The convenience of pouring the concrete mixture, specified in accordance with GOST 6901-54 during its packing with the aid of the standard surface vibrators with a frequency of 2850 vibrations/minute with an amplitude of 0.35 mm can be adopted for the marine reinforced-concrete designs as equalling 15-20 seconds.

In the capacity of an inert filler for the sandy concrete, we use natural sands of average grain size, with a screening out of the particles with a coarseness above the least of the values: one third of the dimension of the mesh of the grids being used, and the thickness of the protective layer of concrete.

The following amount of admixtures in the sand is permitted:

clay and dusty fractions, determined by elutriation, percent by weight.................	not more than 1
sulfuric acid and sulfuric compounds in conversion to SO_3, percent by weight..........	not more than 0.5
shale, opal and other amorphous varieties of silicon................................	not permitted
organic admixtures (colorimetric sample)....	color of solution not darker than the standard according to GOST 8736-58

The recommended granulometric composition of sand:

mesh size of contrillo sieves, mm.........	2.5	1.25	0.63	0.315	0.14
entire residue in sieves, % by weight.....	0	30-40	50-60	65-75	80-90

Based on the studies made, we recommend the following composition of concrete for the shipbuilding reinforced concrete based on Portland-cement, grade 500 (by weight): for the grade 500 concrete, 1:1.5 at V/TS=0.35-0.38; for grade 400 concrete, 1:2 at V/TS=0.35-0.40.

We have presented below the typical differences in the cement-sandy concretes of the indicated compositions from the usual shipbuilding concretes based on a coarse filler (filling agent):

1. The ratio of the prismatic strength to the cubic strength for the sandy concrete is slightly higher than for usual concrete, 0.75-0.80.

2. The resistance of sandy concrete to tension (stretching) is also slightly higher than that of standard concrete of the same grade.

3. The ratio of the resistance of concrete to tension during bending to the resistance during axial tension is higher than for standard concretes, 2.0-2.5.

4. The elasticity modulus of sandy concrete during compression is lower by 20-25% than the standard values of the elasticity modulus for standard concrete.

5. The volumetric weight of sandy concrete (2.2-2.3 t/m^3) is less than the volumetric weight of standard shipbuilding concrete of the same grade.

6. The shrinkage of the cement-sandy concrete is somewhat greater than of the standard, which is explained chiefly by the greater consumption of cement and by the absence of a coarse filler.

7. The creep of the cement-sandy concrete is also greater than of the usual concrete.

The basic physico-mechanical properties of the shipbuilding cement-sandy concrete of the above-recommended compositions, established on the basis of a statistical processing of the results of tests of around 300 samples are presented in Table 2.

Reinforcement. The reinforced-concrete designs are strengthened with thin steel gratings (woven or welded) and with individual rods or with welded rod grids.

Table 2

Physico-Mechanical Properties of Cement-Sandy Concrete

Properties of Concrete	Indexes of Properties of grades of Concrete in Respect to Compressive Strengths*			Samples Being Tested	
	400	500	600	Type	dimensions, cm
Resistance to axial compression not less than, kg/cm^2:					
cubic (grade) strength	400	500	600	cube	7X7X7
prismatic strength	340	420	510	prism	7X7X30
Strength at axial tension not less than, kg/cm^2	30	35	39	cube or beam	GOST 1150-64
Tensile strength during flexure not less than, kg/cm^2	60	70	78	beam	7X7X30
Watertightness at maximum water pressure not less than, kg/cm^2:					
for marine ships	2.5	for all brands		cylinder	15 diameter; 2.5 height
for river ships	2.0	for all brands			2.5 height
Resistance of concrete to freezing in fresh water (for river vessels), in sea water (for marine ships), cycles	50 - 300**			cube	7X7X7

* For the designs operating mainly under tension, with special justification, we permit the additional use of a grade of concrete in respect to tensile strength according to SNIP II-A-10-62.

** The index of the concrete's resistance to freezing is established in dependence on the climatic conditions of the operating region of the ship.

The type of the thin steel grids for reinforcing the marine reinforced-concrete designs is chosen with consideration of getting the maximum possible dispersity of reinforcement (maximal surface of the grid wires) at the required factor of reinforcing the elements of the designs, and the technology of manufacturing them.

For obtaining the maximum dispersity of reinforcement, it is efficient to utilize the gratings with small meshes and made of finer wire, whereas a better packing of the concrete is achieved with grids having the larger meshes.

The investigations of the physico-mechanical properties of shipbuilding reinforced-concrete, reinforced with grids of varying mesh size (3, 5, 6, 7, 8, 9, 10, 11, 12 mm) at uniform consumption of metal and with the identical method of pouring and packing the concrete mixture, with allowance for the labor involved in preparing the designs, demonstrated that for the ship designs, most acceptable are the steel grids with meshes having a size ranging from 5 to 10 mm. Such grids are produced by industry in accordance with State Standard (GOST) 3826-47 (Table 3).

The production of the welded wire grids has not yet been mastered by industry. Their application in place of the wire ones will permit us to raise the stability of the elastic-strength characteristics of the reinforced-concrete, the technological effectiveness of producing the marine designs, and a reduction in their cost. Therefore, the transition to the welded grids (gratings) will be a progressive step on the way to improving the shipbuilding reinforced concrete as a construction material.

In accordance with GOST 3826-47, the fabric nets are made from low-carbon annealed wire, having a considerable spread in its strength characteristics. The expansion diagram of the grids' wire does not have a clearly expressed area of yield.

Table 3

Characteristics of the Wire Grids Recommended for the Shipbuilding Reinforced Concrete (According to GOST 3826-47)

Number of woven mesh	Dimension of mesh cells in light, mm	Diameter of wire mesh	Number of rods (longitudinal and transverse) per sq. m of mesh	Specific surface obtained during saturation of element with thickness of 1 cm of 1 fiber mesh, cm^2/cm^3	Reinforcing factor in 1 direction, obtained during saturation of element (1 cm) by 1 fiber mesh	Weight of 1 sq m of mesh, kg
5	5 X 5	0.7	350	0.770	0.00672	1.1
6	6 X 6	0.7	300	0.660	0.00575	0.9
7	7 X 7	0.7	260	0.572	0.00500	0.8
8	8 X 8	0.7	230	0.506	0.00441	0.7
9	9 X 9	1.0	200	0.628	0.00785	1.3
10	10 X 10	1.0	180	0.570	0.00715	1.2

The tests conducted on individual fabric grids (and also groups of them) for axial expansion conducted during a study of the properties of the shipbuilding reinforced-concrete indicated that the strength of the grids is less than the total strength of the individual wires. The indicated circumstance is explained by the difficulty of accomplishing a uniform stretching of all the wires in the composition of a bundle of fabric grids. Proceeding from this, we have listed below the design characteristics of resistance in the fabric grids recommended for shipbuilding reinforced-concrete:

Diameter of grid wires, mm 1.0 0.7
Design resistance, kg/cm^2 .... 2100 2400

The steel low-carbon wire which is being applied for the additional reinforcement of the extended zone of the reinforced-concrete designs or for the replacement of a part of the fine fabric grids should meet the specifications of GOST 6727-53. The frameworks of the beams in the set and in the various supporting parts of the hull are reinforced by hot-rolled reinforcing metal, meeting the requirements of GOST 5781-61.

Additives. In order to increase the corrosion resistance of the reinforced concrete designs, it is recommended that the cement-sandy concrete be prepared with the addition of an inhibitor, namely sodium nitrite (according to GOST 6194-52) introduced into the concrete with the water used during manufacture in the amount of 1.5-2% of the cement's weight.

In order to reduce the water requirement of the concrete mixture, and also to improve the basic properties of the sandy concrete (resistance to freezing and water, watertightness), during the production, we introduce into the concrete mixture a sulfite-alcohol residue (GOST 8518-57) in the quantity of 0.1-0.2% of the cement's weight.

Section 3. Effect of Various Structural Factors of the Deformative and Strength Properties of Reinforced Concrete.

Characteristics of degree of saturation and distribution of reinforcement. Under the dispersed arrangement of reinforcement metal in the body of the concrete, the forces of adhesion of the reinforcing material with the concrete is considerably more than in the case of concentrated reinforcement. They (the forces) increase in proportion to the surface of the wires of the fabric grids (areas of adhesion), which in the reinforced-concrete element at the given reinforcement factor change in relationship to the diameter of the wires and the size of the grid meshes.

If the diameter of the grid wires remains constant, and only the value (size) of the grid meshes changes, between the adhesion surface and the

reinforcement factor, a single-valued conformity exists, i.e., in these cases the adhesion surface is a universal characteristic of the reinforcement. Thus, the transition from mesh number 5 to mesh number 8 leads to a reduction by 1.5 times of the adhesion surface, and to a reduction by the same number of times, of the reinforcement factor. The single-valued conformity also takes place between the quantity of grids of the same number per unit of element's thickness, surface of adhesion, and reinforcement factor.

At a change in the diameter of the grid wires, the reinforcement factor changes more rapidly than does the adhesion surface, since the reinforcement factor is proportional to the square of the wires' diameter, while the adhesion surface is proportional to the first degree of the wires' diameter. In a most general case of reinforcement under the application of fabric grids made of wire having various diameters, in combination with the rod reinforcement, the basic characteristic of reinforcement of concrete, just as of standard ferroconcrete, is the reinforcement factor μ, the value of which provides a complete concept concerning the extent of saturation of the concrete body by the steel reinforcement and establishes the critical carrying capacity of reinforced concrete (as of ferroconcrete in general) under tension. The adhesion surface of the reinforcing material with the concrete in the general case of reinforcement under consideration loses its universality and constitutes only an index of the degree of dispersion of the reinforcement material in the concrete body.

The degree of dispersity of reinforcement is conventionally expressed by the so-called specific surface of reinforcement (total surface of the wire of metal grids in a unit of the reinforced-concrete element's volume), signified by K_{η} and having the dimensionality cm^2/cm^3.

To establish the degree of effect of the value of critical surface of the reinforcement and the value of the reinforcement factor on the nature of deformation, cracked formation and strength of reinforced concrete, we have conducted a series of experimental studies; their results and analysis are presented below. The experimental studies were conducted on samples and designs, in which the age of the concrete was not less than 1.5-3 months, which corresponds to the age of the concrete in the designs up to the time of delivering the ships for operation.

Effect of specific surface and reinforcement factor on the deformative and strength properties of reinforced concrete. Since for grids with wire of the same diameter, a single-valued conformity occurs between the specific reinforcing surface K_n and the reinforcing factor μ, it does not appear possible to reveal the separate influence of each of these characteristics of the fabric grids upon the stress-strain state of reinforced concrete under the condition of reinforcing it by grids containing wire of the same diameter. In connection with this, and also for expanding the range of variations in K_n at μ = const and variations in μ at K_n = const in the production of the experimental models, we utilized, in addition to the grids number 5-8 with a wire diameter of 0.7 mm and number 9-10 with a wire diameter of 1.0 mm, the grids number 3, 2 with a wire diameter of 0.45 mm and grids number 11 and 12 with a wire diameter of 1.2 mm. In this connection, the values for the reinforcing factors varied in the limits μ = (1.75 - 2.8%), which corresponds to the lower and upper limits of the degree of saturation, by metal, of the marine reinforced-concrete designs (respectively 300 and 450 kg per cubic meter of ferroconcrete).

The studies were conducted for the cases of axial expansion and bending of the reinforced concrete elements (Tables 4 and 5), since under these types of load, there was manifested most clearly the effect of the dispersity of reinforcement upon the nature of the deformation of the concrete.

As follows from the tables, the perceptible difference in the values of the average relative deformations, corresponding to the appearance of visible cracks in the reinforced concrete, as compared with standard ferroconcrete, takes place only at $K_{\eta} = 2\ cm^2/cm^3$.

With an increase in the degree of dispersion of reinforcement, K_{η}, other conditions being equal (of the constants of the reinforcement factor and strength of concrete), the average relative stresses, corresponding to the appearance of visible cracks, increase. This is explained by virtue of the fact that at an increase in the specific surface of reinforcement, K_{η}, the adhesion forces of the reinforcement with the concrete, referred to the area of the concrete's cross section, also increase, owing to which the moment is separated, corresponding to the opening of the cracks by the identical amount. This result confirms the known concept of the theory of ferroconcrete to the effect that the width of the cracks' opening, at identical expenditures of metal and identical loads on a design, is determined by the nature of distribution of the reinforcement in the concrete's body. In this connection, the relative elongations, with allowance for the crack formation during axial expansion proved to be about half as much as during the pure flexure (for the extremely expanded fibers). The values of the loads and hence of the stresses, just as the values of the relative stresses during the crack formation in the reinforced concrete, are determined by the values of the reinforcement factor μ and by the specific surface of the reinforcement, K_{η}.

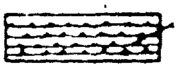
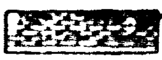
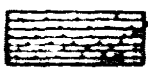
At the identical reinforcement factor μ = const, as follows from the data presented in Tables 4 and 5, the stresses and the relative elongations during the crack formation in the reinforced concrete increase with an increase in K_{η} to a specific limit, equalling 3.0-3.5 cm^2/cm^3. From a comparison of the data given for the samples in series VII with the data for the samples in

series IV and V (Table 4) and the data for the samples in series XIV with the data for the samples in the series XI and XII (Table 5), having essentially the same reinforcement factors and equal strengths of concrete, it follows that an increase in the specific surface of reinforcement above the indicated critical value leads to a decrease in the stresses of crack formation in the reinforced concrete. This is explained by the insufficient packing of the concrete mixture in the reinforced concrete element in the case of its excessive saturation by metal evenly distributed through the entire section. As a result, the wires in the grids are insufficiently covered with the concrete layer required for providing the combined action of the grids and concrete, and upon the effect of a load upon such an element, it divides into a layer. Since the increase in the stresses of crack formation with an increase in K_Π takes place less intensively than the increase in the average relative elongations with allowance for crack formation (within the limits of the limited width of cracks' opening), the increase in K_Π at μ = const leads to a certain reduction in the normal modulus of the material with allowance for the crack formation taking place in it. From a quantitative standpoint, the conditions noted are characterized by the data given in Tables 4 and 5.

At the same specific surface of reinforcement K_Π = const, with an increase in the reinforcement factor, μ, we have an increase in the stresses and in the average relative elongations of crack formation. In this connection, the increase in μ is reflected to a much greater extent upon the increase in the stresses of crack formation than on the increments of the average relative elongations during the crack formation. This is quite regular, since the warping tendency, with allowance for the opening of the cracks, is determined chiefly by the specific adhesion surface of the reinforcement material with the concrete, and an increase in μ at maintenance of K_Π = const is equivalent to an increase in the reduced area of the reinforced concrete sample.

Table 4

Results of Testing the Samples for Axial Expansion

Number of series	Number of samples in series	System of reinforcement (cross section)	Dimensions of Samples, mm	Strength of concrete* during tests of sample, kg/cm²	Specific surface of reinforcement K_n, cm²/cm³	Reinforcement factor μ, in cross section, %	Stress of crack formation** σ_{tp}, kg/cm²	Breakdown strain $\sigma_{p.s.p.}$, kg/cm²	Relative elongation of crack formation** $\varepsilon_{tp} \cdot 10^{5}$	Secant modulus of crack formation** $E_{tp} = \sigma_{tp}/\varepsilon_{tp}$, kg/cm²
I	6	 4 mesh N 11	1000×80×20	[illegible]	1,2	1,86	[illegible]	48	[illegible]	86 000
II	6	 8 mesh N 8	1000×80×20	[illegible]	2,08	1,8	42	48	55	76 000
III	6	 8 mesh N 11	1000×80×20	[illegible]	3,04	1,75	45	50	70	64 000
IV	6	 6 mesh N 11	1000×[illegible]×[illegible]	[illegible]	1,[illegible]	2,8	[illegible]	80	60	96 000
V	6	 12 mesh N 7	1000×80×20	[illegible]	3,18	2,7	73	80	100	73 000
VI	6	 12 mesh N 8	1000×80×20	730	3,12	2,7	82	90	100	82 000
VII	6	 N 11	1000×80×20	[illegible]	4,[illegible]	2,[illegible]	80	[illegible]	100	80 000

* Strength of concrete is established for blocks 7 X 7 X 7 cm.

** Crack formation signifies the opening of cracks on sample's surface by distance of 0.04 - 0.05 mm.

As a result, with an increase in μ at K_n = const, a certain increase in the standard modulus occurs.

As the studies conducted have indicated, the destruction of all the samples tested started with the rupture of the most elongated grids. In this connection, the braking load for all the samples having equal reinforcement factors, was practically identical, and did not depend on the size of the specific surface. This provides evidence to the effect that the critical state of the reinforced concrete in respect to strength during stretching and bending is determined only by the quantity of reinforcing metal, independently of the extent of its dispersity in the body of the concrete.

An analysis of the data given in Tables 4 and 5 permits us to formulate the conditions of the most efficient choice of reinforcement from the viewpoint of crack resistance and strength of the reinforced-concrete design:

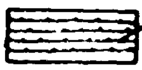
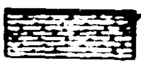
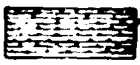
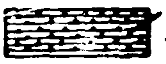
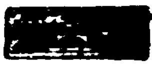
1. The dispersity of the reinforcement, determining the warping tendency of the reinforced concrete, and hence the degree of utilization of the reinforcement should be that which is maximally permissible.

2. The quantity of reinforcement should be that which is minimally required, from the condition of assurring the required carrying capacity of the reinforced-concrete designs in respect to strength.

Effect of Reinforcing Rods Upon the Deformative and Strength Properties of Reinforced Concrete. We examined previously the reinforcement of cement-sandy concrete only by wire steel grids. We indicated that the area of the specific reinforcing surface for the marine reinforced-concrete designs should comprise 2-3 cm^2/cm^3.

Based on the data in Table 3, we can establish that for attaining the dispersity of reinforcement with K_n = 2-3 cm^2/cm^3, for 1 cm of an element's thickness, it is necessary to install the grids:

Results Obtained from Testing Samples for Pure Flexure

Number of series	Number of samples in series	Number of reinforcement (cross section)	Sizes of samples, mm	Strength of concrete* during testing of sample, kg/cm²	Specific surface of reinforcement, K_n, cm²/cm³	Reinforcement factor μ, in one direction, %	Crack formation stresses** $\sigma_{Tp} = M/W$, kg/cm²	Relative elongations during crack formation: Of expanded fiber $\epsilon^{p}_{Tp} \cdot 10^{5}$	Relative elongations during crack formation: Of compressed fiber $\epsilon^{cm}_{Tp} \cdot 10^{5}$	Strain modulus of crack formation**: In expanded zone $E^{p} = \frac{\sigma_{Tp}}{\epsilon^{p}_{Tp}}$, kg/cm²	Strain modulus of crack formation**: In compressed zone $E^{cm} = \frac{\sigma_{Tp}}{\epsilon^{cm}_{Tp}}$, kg/cm²
VIII	6	4 mesh  N 11	1000×80×20	650	1,2	1,85	100	110	55	91 000	182 000
IX	6	8 mesh  N 8	1000×80×20	650	2,08	1,8	120	150	70	80 000	171 000
X	6	8 mesh  N 12	1000×80×20	650	04	1,75	130	170	100	76 000	130 000
XI	6	6 mesh  N 11	1000×80×20	680	1,8	2,8	160	150	65	107 000	246 000
XII	6	12 mesh  N 8	1000×80×20	650	3,12	2,7	190	215	90	88 000	210 000
XIII	6	12 mesh  N 8	1000×80×20	750	5,12	2,7	215	230	100	94 000	215 000
XIV	6	12 mesh  N 11	1000×80×20	680	4,80	2,66	180	225	95	83 000	126 000

* Strength of concrete was established for blocks 7 X 7 X 7 cm.

** Crack formation signifies the cracks' opening on the sample's surface by the amount of 0.04–0.05 mm. The stresses determined from formula $\sigma = M/W$ are provisional. The true stress values are obtained if necessary from the equilibrium condition with allowance for the difference in elastic characteristics in the expanded and compressed zones (see Sect. 21).

No. 5......................	from 3 to 4
No. 6......................	from 3 to 4
No. 7......................	from 4 to 5
No. 8......................	from 4 to 6
No. 9......................	from 3 to 4
No. 10.....................	from 4 to 5

Designs have also appeared with combined reinforcement, in which a part of the wire grids in the center of the section height is replaced by reinforcing rod or by rod-type grids.

An evaluation of the effect of the intermediate rod-type grid or of individual rods situated in the central part of the height of the section of reinforced concrete element for the elastic-strength characteristics of reinforced concrete during bending and axial stretching was conducted on the samples, the characteristics of which are shown in Tables 6 and 7.

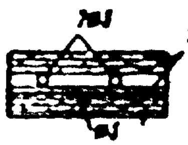
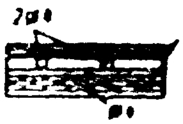
An analysis of the data presented in Tables 6 and 7 permits us to make the following conclusions:

1. The average relative elongations with allowance for the crack formation during pure bending for the samples reinforced only with fabric grids and for the samples, in which the part of fabric grids is replaced by rod reinforcement or by rod-type grids are practically identical at a constant specific reinforcement surface K_{η} in the extreme fibers, and do not depend on the reinforcement factor, μ.

The difference in the relative elongations of crack formation for the samples in the XIX and XXII series (see Table 7) and for the samples in the XI series (see Table 5), having practically identical specific reinforcement surfaces, K_{η} and reinforcement factors μ, is explained by the fact that the data for the series XIX and XXII pertain to the moment of the appearance of the samples' surface) of cracks with an opening size of 0.01 mm, while the data for the XI series pertain to the moment of the appearance (on the surface of the samples)

Table 6

Results of Comparative Tests for Axial Expansion of Samples with Combined Reinforcement and of Samples Reinforced Only with Woven Screent

Number of series	Number of samples in series	System of reinforcement (cross section)	Dimensions (mm) of samples	Strength of concrete during tests of sample*, kg/cm²	Specific surface K_n of reinforcement, cm²/cm³		Reinforcement factor μ, in one direction %		Strain of crack formation**, $\sigma_{T p}$, kg/cm²	Breaking strain ($\sigma_{p.p}$), kg/cm²	Relative Elongation of crack formation** $\varepsilon_{Tp} \cdot 10^5$	Modulus of crack formation, $E_{Tp} = \frac{\sigma_T}{\varepsilon_{Tp}}$, kg/cm²
					In cross fibers	Total	With meshes	Total				
XV	3	 9 mesh N5	1200×180×30	780	2,88	2,32	2,02	2,02	63	68	60	89 000
XVI	3	 7Ø3 3 meshes N5 Ø3	1200×150×30	780	2,32	1,79	1,34	2,21	42	88	45	92 000
XVII	3	 10 meshes N8	900×102×20	680	2,62	2,62	2,2	2,2	66	—	66	100 000
XVIII	3	 7Ø4 N8 Ø4	600×103×21	600	2,82	1,79	1,32	2,64	54	—	45	120 000

* Strength of concrete was established for blocks 7 X 7 X 7 cm.

**Crack formation for samples in series XV and XVI signifies the opening of cracks on the samples' surface by 0.01 mm; for the samples in series XVII and XVIII, by 0.025 mm.

of the cracks with an opening size of 0.04-0.05 mm. In addition, the strength of the concrete up to the time of the testing of samples indicated in Table 7 is appreciably less than the strength of the concrete in the samples listed in Table 5.

2. The average relative elongations with allowance for the crack formation during axial expansion (stretching) for the specimens reinforced only with the fabric grids are greater than for the samples having combined reinforcement. In this connection, it is worth noting that the first cracks appear in the sections above the transverse rod reinforcement. This is explained by the disruption of the structure of the dispersed reinforcement in the central part of the samples' cross section.

The difference in the stresses (deformations) of crack formation in samples of the XV series (Table 6) and of samples of the II series (Table 4), having close values of specific surface and reinforcement factors are explained (as also in the bending) by the varying value of cracks' opening and by the differing strength of the concrete in these samples.

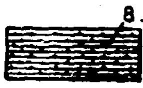
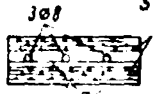
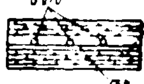
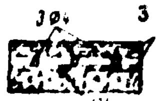
3. In the case of bending (flexure) for the samples with combined reinforcement, we find the same qualitative dependences of the modulus of stresses upon the reinforcement factor μ and the specific reinforcement surface K_{η} in the extreme fibers as for the samples reinforced only by the fabric gratings (grids).

4. The braking load for both types of reinforcement is determined by the value of the reinforcement factor μ and does not depend on the degree of reinforcement dispersity, K_{η}.

Effect of Grade of Concrete and Depth of Protective Concrete Layer Upon the Deformative and Strength Properties of Reinforced Concrete. Since the bearing capacity of the reinforced-concrete elements, operating under expansion and

Table 7

Results of Comparative Tests for Pure Bending of Samples Reinforced Only with Woven Grids and of Samples in Which Part of the Woven Grids is Replaced by Rod-type Reinforcement

Number of series	Number of samples in series	Reinforcing system (cross section)	Sample sizes, mm	Strength of concrete during testing samples*, kg/cm²	Specific surface K_n of reinforcement, cm²/cm³: in extreme fibers	Specific surface K_n of reinforcement, cm²/cm³: total	Reinforcement factor μ, in one direction, %: with meshes	Reinforcement factor μ, in one direction, %: total	Stress of crack formation $\sigma_{rp} = \frac{M}{W}$, kg/cm²	Relative elongations during crack formation**: of expanded fiber $\epsilon^{p}_{rp} \cdot 10^{5}$	Relative elongations during crack formation**: of compressed fibers $\epsilon^{cm}_{rp} \cdot 10^{5}$	Strain modulus of crack formation**, kg/cm²: in expanded zone $E^{p} = \frac{\sigma_{rp}}{\epsilon^{p}_{rp}}$	Strain modulus of crack formation**, kg/cm²: in contracted zone $E^{cm} = \frac{\sigma_{rp}}{\epsilon^{cm}_{p}}$
XIX	3	 8 mesh N12	[illegible] × [illegible] × 25	[illegible]	1.[illegible]	1.83	2.74	2.74	[illegible]	95	[illegible]	[illegible] 000	155 000
XX	3	 3ø8; 2 meshes each N8; ø8	[illegible] × [illegible] × 25	[illegible]	[illegible]	0.97	0.71	2.07	70	100	[illegible]	70 000	120 000
XXI	3	 ø4; 3 meshes each N12	1000 × 250 × 25	300	1.[illegible]	1.42	2.05	2.06	70	100	50	70 000	120 000
XXII	3	 3ø8; 3 meshes each N8; ø5	1000 × 250 × 25	300	1.9	1.17	0.88	2.90	90	100	60	90 000	150 000
XXIII	3	 3ø6; 3 meshes each N8; ø4	1000 × 250 × 28	300	2.02	1.35	1.06	2.38	90	115	70	78 000	129 000
XXIV	3	 10 meshes N8	1000 × 184 × 21	400	2.52	2.52	2.2	2.20	100	145	65	69 000	154 [illegible]
XXV	3	 3ø4; 3 meshes each N8; ø4	1000 × 181 × 23	400	[illegible]	1.75	1.46	2.82	[illegible]	120	55	79 000	173 000

* Strength for samples in series XIX-XXIII were established for blocks 10 X 10 X 10 cm; for samples in series XXIV-XXV- for blocks 7 X 7 X 7 cm.

** The crack formation for samples in series XIX-XXIII signifies the cracks' opening on the samples' surface by 0.01 mm; for samples in series XXIV-XXV- by 0.025 mm.

bending, is determined only by the amount of reinforcing metal, we can speak of the effect of the grade and depth of the protective concrete layer only in application to the relative stresses of the reinforced concrete's crack formation.

It is known that the limit of the resistance of concrete to expansion and its maximum ductility at a significant increase in the quality of the concrete increase slightly, i.e. the ratio R_p/R decreases with an increase in the quality (grade). In this connection, a decisive effect is exerted on the extent of the maximum ductility of concrete by the degree of the concrete's uniformity.

This widely known concept has been confirmed also in a study of the qualities of reinforced concrete, specifically in the transition from a concrete strength of 650 kg/cm^2 to a concrete strength of 760 kg/cm^2, other conditions being equal, the forces of crack formation in the reinforced-concrete samples differ by not more than 10% (refer to Tables 4 and 5).

The bearing capacity of reinforced concrete elements functioning under compression is established chiefly by the resistance of concrete during compression, in connection with which the strength of such elements increases with an increase in the quality of the concrete.

Proceeding from the condition of preservation of the constant degree of dispersity of reinforcement through the entire section of the element, the value of the concrete protective layer in the reinforced-concrete designs should comprise 2-3 mm. An increase in the protective layer of concrete to the values adopted in the usual ferroconcrete designs (10 mm and more) leads to a change in the nature of the crack formation in the reinforced-concrete elements operating under expansion: to an increase in the width of the cracks' opening, at the simultaneous increase in the pitch of the cracks. The occurrence indicated

can be explained by the disruption of the dispersity of reinforcement in the extreme fibers of the reinforced concrete elements.

The survey and the analysis given in Tables 4, 5, 6 and 7 permit us to note that from the viewpoint of the expenditure of metal, the technology of production and the obtainment of sufficiently high elastic-strength characteristics during the axial expansion and bending, the most optimal systems of reinforcement are:

--for reinforcement only by fabric grids--mesh No. 8 in the quantity of 4-5 per cm of element's thickness; and

--for the combined reinforcement--the rod-type grid with a wire diameter of 4-5 mm and the fabric mesh No. 8 from the calculation of 4-5 items per centimeter of height of the dispersed-reinforced extreme fibers.

In this connection, the cement-sandy concrete for the production of the marine designs of reinforced concrete should be of grade 400-500.

Section 4. Strain State and Crack Formation of Reinforced Concrete During Axial Expansion and Bending

Above, based on an analysis of the effect of the various structural factors on the mechanical properties of reinforced concrete and a consideration of the technological effectiveness of producing the reinforced-concrete designs, we establish the most optimal types of reinforcement, grade of concrete and depth of the protective layer of concrete (series XVII, XVIII, XXIV and XXV, Table 6).

Now let us examine the nature of the deformation and crack formation in shipbuilding reinforced concrete at all stages of loading all the way to the braking of the samples.

Axial Expansion and Expansion During Bending. The stress-strain diagrams during axial expansion and expansion during bending for the samples in series

XVII, XVIII, XXIV and XXV are shown in Figs. 6 and 7, while the load-sagging relationship is indicated in Fig. 8. A review of the stress-strain diagrams for the cases of axial expansion and expansion during flexure permits us to identify three typical sectors.

I. Sector of Elastic Stresses of Reinforced Concrete [relative stresses $\varepsilon = (0\text{-}15) \cdot 10^{-5}$ for axial expansion and $\varepsilon = (0\text{-}25) \cdot 10^{-5}$ for expansion during bending.] In this sector, the diagram in its outline is close to the diagram of nonreinforced concrete, and, with a certain assumption, can be taken as rectilinear.

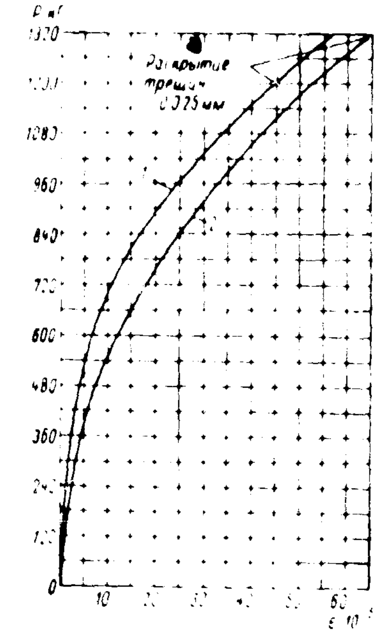


Fig. 6. Average Curves of Dependence of Relative Stresses Upon Load During Axial Expansion. 1- samples with combined reinforcement (XVIII series); 2- samples reinforced only by fabric gratings (series XVII). Key: a) opening of cracks 0,025 mm.

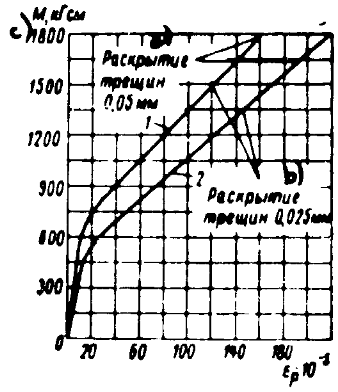


Fig. 7. **Averaged Curves of the Dependence of Relative Stresses of the Extreme Extended Fibers Upon Load, During Pure Flexure.** 1- samples with combined reinforcement (series XXV); 2- samples reinforced only by fabric grids (series XXIV). Key: a) opening of cracks 0.05 mm; b) opening of cracks 0.025 mm; and c) M_f kg/cm.

II. **Sector of Origination and Development of Microcracks** [relative stresses $\varepsilon = (15\text{-}20) \cdot 10^{-5}$ for axial expansion and $\varepsilon = (25\text{-}30) \cdot 10^{-5}$ for expansion during flexure]. In this sector of the diagram, the lines indicating the functional dependence of the relative stresses upon load are distorted, which testifies to the structural changes, occurring in the reinforced concrete, and caused by the process of microcrack formation. In this connection, we did not succeed in detecting the microcracks with the naked eye; however, their presence was established by strain gauge sensing elements, and also with a microscope with a 70-X magnification. A number of other researchers for this /35 range of deformations (strains) also established the presence of microcracks with the aid of ultrasonic equipment. Such microcracks, at a high degree of dispersity of reinforcement of the concrete, in spite of their force origin, in effect do not disrupt the continuity of the reinforced concrete as a material, and have rather a theoretical than a practical importance. In its nature, in general concrete has a heterogeneous structure, i.e., in it prior to the application of loads, we can find flaws, similar to cracks. The influence of the structural flaws in the concrete upon the strength of the designs depends on the conditions in which the concrete is used.

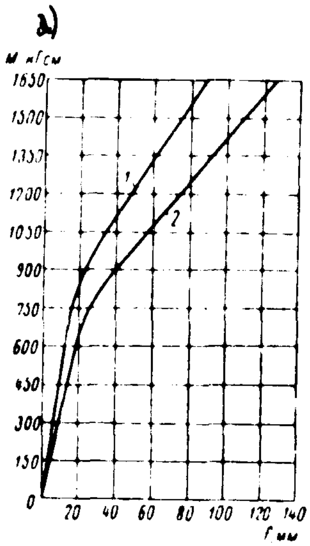


Fig. 8. Averaged Curves of the Dependence of Saggings Upon Load During Pure Flexure. 1- samples with combined reinforcement (series XXV); and 2- samples reinforced only by fabric screens (series XXIV). Key: a) M, kg/cm.

In the case of unreinforced concrete, the microcracks (both of a structural and of a strength origin), constituting concentrators of strains, become a source of brittle breakdown of the concrete under the loads, corresponding to the attainment by the concrete of the temporary resistance to expansion, R_p. In the case of concentrated reinforcement (standard ferroconcrete) as a result of the uneven redistribution of forces through the space between the concrete and reinforcement after the development of microcracks in the concrete, the opening of individual cracks perceptible to the naked eye occurs.

However, in the case of the dispersed reinforcement (reinforced concrete), in view of the large adhesion surface of the reinforcement with the concrete, and on the basis of this redistribution (more uniform through the space) of forces between the reinforcement and the concrete, after the development of the microcracks in the concrete, the cracks under the stresses equalling the strength limit of the concrete to expansion and even exceeding this limit slightly, do not transfer to the category of cracks visible to the naked eye. Such microcracks in the reinforced concrete in essence, not withstanding their force origin, can be likened to the microcracks of a structural nature

in the concrete. In this connection, the process of microcrack formation for the extent of the entire sector II of the diagram takes place chiefly in the direction of an increase in the number of microcracks, and not in the width of their opening. This assures the functioning of the concrete in reinforced concrete in sector II of the diagram in the elastic-plastic stage without a significant breakdown in the solid state of the material, whereas in standard ferroconcrete under the same stresses (close to R_p), we find visible cracks, entirely disrupting the solid state of the concrete. /36

III. **Sector of Formation of Visible Cracks.** At the relative elongations $\varepsilon = (20\text{-}30) \cdot 10^{-5}$, the dispersed-reinforced concrete is deformed plastically, the volume of the microcracks in the concrete gradually increases, and after reaching a critical value, on the surface of the reinforced-concrete samples, visible cracks appear. This is evidence to the effect that also under the conditions of dispersed reinforcement, the plastic deformations possible for realization prove to be fully exhausted. It should be noted that in the samples with the combined reinforcement, the first visible cracks appear in the sections above the transverse rod-type reinforcement, moreover, they open immediately by a space of about 0.01 mm. At the same time, the initial opening of the visible cracks in the samples reinforced only by the fabric screen comprises a value of the order of 0.003-0.005 mm.

Since the extent of the opening of the first visible cracks for the different types of reinforcement is different, for the possibility of comparison, the elastic-strength characteristics of both types of reinforced concrete pertain to the moment, corresponding to the uniform opening of cracks (0.01, 0.025 or 0.05 mm). In this connection, the average relative elongations of reinforced concrete for two of its variants attain the values: $\varepsilon = (120\text{-}150) \cdot 10^{-5}$ for the extended fibers of the elements which are being bent and $\varepsilon = (60\text{-}70) \cdot 10^{-5}$ for the elements subjected to the axial expansion.

The sector of the stress-strain diagram under review is typical with the functioning of the reinforcement in the zone of elastic strains and functioning of concrete in the zone of plastic deformations. The combined functioning of the concrete and the reinforcement is not disrupted in this connection, and in sector III of the diagram, the reinforced concrete can be tentatively considered as a material without disruptions of its continuity.

The diagram for the sector in question has a practically rectilinear nature, which is explained by the functioning of the reinforcement in the zone of elastic deformations and by the slight influence of the plastic expansion of concrete, penetrated by microcracks, upon the nature of the stress-strain relationship.

The load corresponding to the opening of the visible cracks by the distance 0.01-0.05 mm, comprises 0.75-0.80 of the destructive load. Here it is convenient to note that the ratio of the load corresponding to the opening of the cracks by the amount of 0.01-0.05 mm to the destructive amount for standard ferroconcrete comprises a value of only 0.35-0.50. The relationships indicated confirm the favorable influence of the distribution of reinforcement upon the phenomenon of crack formation in concrete.

The dispersed reinforcement introduces definite features into the kinetics of the crack formation process also. Thus, while the disruption of the elements made of standard ferroconcrete precedes the appearance and the opening of one or a small number of cracks, in the reinforced concrete, with an increase in the load, after the appearance of visible cracks, the development of additional strains takes place not so much owing to the opening of the cracks which initially appeared as owing to the appearance of the new cracks. The openings of the cracks which appeared originally in effect do not occur until the entire surface of the sample is covered by a network of cracks with /37

a step, equalling 1-2 meshes of the wire grid. The opening of the originally developed cracks is delayed by the presence of the high adhesion forces of the reinforcing material and the concrete. At a further increase in the load in the range directly preceding breakdown, we find an intensive opening of many cracks. Up to the moment of breakdown, the working surface of the sample is covered with a solid network of cracks, moreover the width of the cracks' opening on the surface of the part comprises 0.1-0.2 mm (Fig. 9).

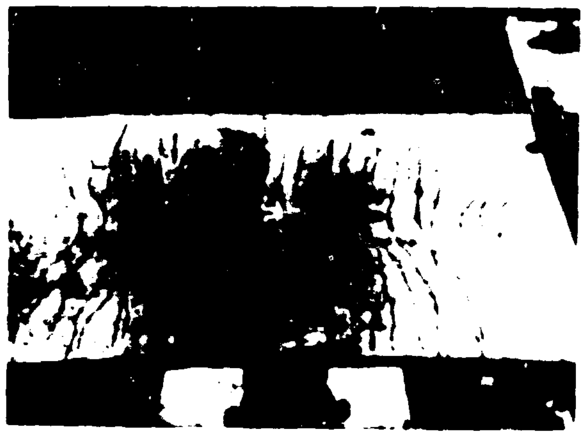


Fig. 9. View of the Expanded Surface of a Reinforced-Concrete Sample During a Test for Pure Flexure.

The nature of the deformation of reinforced concrete during axial expansion and expansion during bending is essentially the same both for the case of reinforcement only by fabric nets and for the case of combined reinforcement. All of the typical sectors of the stress-strain diagram which we have reviewed, and the qualitative phenomena accompanying them take place in both instances. The difference resides only in the quantitative characteristics establishing /38 the limit of the sectors for a given type of reinforced concrete. These quantitative characteristics are presented graphically in Figs. 6 and 7. We should also turn attention to the fact of the difference in the elasticity moduli of reinforced concrete at the various stages of its stress-strain condition (state).

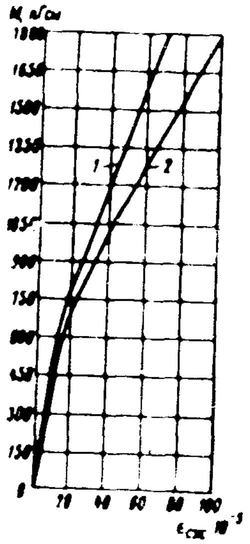


Fig. 10. Averaged Curves of the Dependence of Relative Strains of Extreme Compressed Fibers Upon Load During Pure Flexure. 1- samples with combined reinforcement (XXV series); and 2- samples reinforced only by fabric grids (XXIV series). Key: a) M, kg/cm

Thus, as follows from Figs. 6 and 7, sectors I and III, with accuracy sufficient for the practice, can be approximated by segments of straight lines.

Sector II has a curvilinear outline. However, if we take into consideration that the calculated stress-strain states of the marine concrete reinforced designs (both in the operating stage as well as in the critical state in respect to strength) will correspond to the sector III of the stres... sin diagram, for planning, it is sufficient to have the value of the el-sti...y modulus only in this sector. The values of the elasticity moduli of reinforced concrete for the various systems of reinforcement are presented in Tables 4, 5, 6 and 7, where it is indicated that the value of the elasticity modulus corresponds to that stress-strain state of reinforced concrete at which on its surface we find cracks with an opening distance of .01- .05 mm. This corresponds to the sector III of the stress-strain diagram.

Compression During Bending. During the testing for pure bending of reinforced-concrete samples with different types of reinforcement (Tables 5 and 7),

in the compressed zone of the samples, we did not observe any visible structural changes (of cracks, cleavages, etc.) all the way to breakdown. The disruption of the compressed zone has a brittle nature and develops after the breakage of the extreme meshes of the extended zone. In this connection, in the compressed zone, there occurs a cleavage of the protective layer of concrete and a crumpling of the bare meshes.

Based on the results of testing the samples in series XXIV and XXV, we have compiled the stress-strain diagram for the extreme compressed fibers (Fig. 10).

An examination of Tables 5 and 7, the diagram and the nature of the breakdown of the compressed zone of the flexed reinforced-concrete samples permits us to note the following.

The relative strains of the compressed zone follow the strains of the elongated zone in conformity with the distribution of the internal forces through the height of the section for the unreinforced concrete. The value of the relative strains of the compressed zone through the entire range of loads (from zero to breakdown) comprises 0.4-0.5 of the value of relative strains of the extended zone. In connection with , the relationship of the relative strains in the compressed zone of the parts which are being bent, to the degree of dispersion of reinforcement K_n, which could have been established based on the data in Tables 5 and7, would have had a purely formal nature. In reality, the specific surface of the reinforcement K_n at its maximally permissible values in effect does not exert any influence upon the deformability of the compressed zone of reinforced concrete during bending, and the latter is determined by the deformability of the concrete. /39

However, in the case of the exceeding, by the specific reinforcement surface, of the possibly permissible limit (e.g., at K_n = 4.56 cm^2/cm^3), the

woven wire gauzes deteriorate the working capacity of the compressed zone, causing it to become stratified.

Section 5. Strength of Reinforced Concrete During Central Compression.

The determination of the resistivity of reinforced concrete to axial compression was conducted on prismatic samples measuring 80 X 60 X 20 mm. The samples were reinforced only with wire gauze (No. 5 and 10). The specific reinforcement surface K_n was altered within the limits of 0.6-3.0 cm^2/cm^3, while the percentage of reinforcement μ was accordingly changed in the limits ranging from 0.7-2.8%. The strength of the sandy concrete, determined for blocks with measurements of 7 X 7 X 7 cm, comprised roughly 400 kg/cm^2. The testing of the samples for axial compression was done with a force acting in a plane, parallel to the wire grids. By the investigations of the properties of high-grade sandy concrete (grades 400 and 500), it was established that for such concretes, the ratio of the prismatic and cube-type strength comprises $R_{\eta\rho}/R = 0.75-0.8$.

The tests conducted on the reinforced prismatic samples indicated that the resistance of reinforced concrete to compression is determined chiefly by the prismatic strength of the concrete. The specific surface of reinforcement and the reinforcement factor do not exert any appreciable influence upon the resistance of reinforced concrete to compression.

The increase in the specific surface of the reinforcement from 1 to 3 cm^2/cm^3 and accordingly the increase in the reinforcement factor from 0.7 to 2.8% yields an increase in the resistance of reinforced concrete to compression only by 15%.

The maximum increment in the resistance of reinforced concrete to compression occurs at an increase in K_n from 2 to 3 cm^2/cm^3, and accordingly an increase in the μ-value from 1.7 to 2.8%. /40

Section 6. Deformability of Reinforced Concrete Under Prolonged Effect of Load

The nature of deformation and crack formation reviewed in Section 4 pertains to the case of a temporary load.

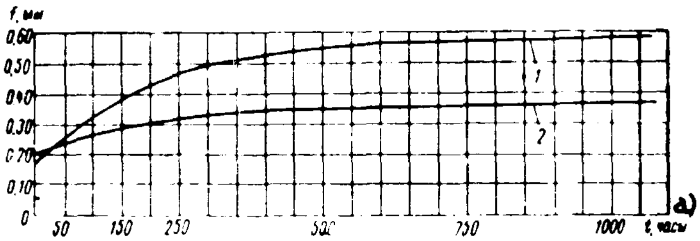


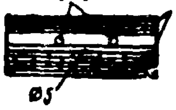
Fig. 11. Curves Depicting Increase in Saggings of Beam-Strips During Prolonged Load According to Pure Bending System.
1 - samples with combined reinforcement; 2 - samples reinforced only with woven meshes. Key: a) t, hours

Taking into account that the basic type of load on hull designs is the flexure in combination with expansion or compression, the deformability of reinforced concrete under the prolonged effect of a load has been studied for the case of pure bending. The tests were run on the beam-strips, having both the combined reinforcement and the reinforcement only by woven meshes. The age of the concrete up to the time of loading the samples was 2 months. In the process of the tests, we measured the value of external load, of saggings, and we also observed the nature of the crack formation, with a measurement of the width of the cracks' opening. In this connection, the constantly acting load on the sample constituted about 0.7 of the temporary load, causing the appearance of cracks with an opening of 0.01 mm. The stresses during this load, computed from the formula $\sigma = M/W$, equalled 50 kg/cm². The characteristics of the samples, the values of the initial and final saggings, and also the strain moduli computed on the basis of the saggings, are listed in Table 8.

As a result of the daily measurements conducted of the saggings in the process of the entire period of holding the samples under a constant load, we constructed a graph for variations in saggings through time (Figure 11).

Table 8

Results Obtained from Tests on Samples During Prolonged Loading According to System of Pure Flexure

Number of series	Number of samples in series	System of reinforcement (cross section)	Sizes of samples, mm	Strength of concrete* during loading of sample, kg/cm²	Specific surface** of reinforcement, K_{Π}, cm²/cm³	Reinforcement factor μ, in one direction, %	Sagging at start of testing, mm	Sagging after 2 months' trial, mm	Modulus of strain based on saggings: at start of test E_p, kg/cm²	Modulus of strain based on saggings: after 2 months' trial, E_p, kg/cm²
I	3		1200 × 150 × 35	600	1,74	1,51	0,20	0,37	2,3·10[illegible]	1,24·10[illegible]
II	3		1200 × 150 × 35	600	1,79	1,76	0,18	0,59	2,45·10[illegible]	0,75·10[illegible]

* Strength of concrete was established for blocks with dimensions of 7 X 7 X 7 cm.

** The value for the specific surface of reinforcement for samples with combined reinforcement pertains to the dispersed-reinforced extreme zones.

The tests were conducted under the conditions of the atmosphere of an enclosed building in which the temperature and air humidity remained practically constant throughout the entire testing period.

/41

As we see from Tables 8 and Fig. 11, in the process of holding the samples under a load, the values of their saggings increased. In this connection, the most intensive increase in the saggings occurred in the first days of keeping the samples under a load; then the intensity of saggings' increase reduced appreciably. After a 2 months' holding of the samples under a load, in effect the stabilization of the saggings developed.

In comparing the behavior of the samples with the combined reinforcement and of the samples reinforced only with woven meshes, it was noted that the increase in the saggings under the effect of a constant load through time, for the samples with the combined reinforcement proceeds more intensively; moreover, the period of intensive increase in the saggings for these specimens is twice as prolonged as for the samples reinforced only with the woven meshes.

A more intensive increase in the deformations through time for the samples with the combined reinforcement, having had immediately after the application of the load the values of saggings and of the reduced strain moduli, close to the values of the same quantities for the samples reinforced only by the woven meshes, is explained by the difference in the nature of crack formation of the reinforced concrete in case of both types of reinforcement.

In reality, as we have indicated in Sections 3 and 4, in the samples with the combined reinforcement, the first cracks, appearing and located in the sections including the transverse rod-type reinforcement, open at once by 0.01 mm. At the same time, the initial opening of the cracks appearing in the samples reinforced only by the woven meshes, comprises 0.003-0.005 mm.

The same situation also held true in the prolonged experiments. In these

tests, the stiffness of the samples with the composite reinforcement decreased more abruptly, and hence in them the intensity of the saggings' increase was also greater, wherein the width of opening of the cracks which had appeared in the course of the entire testing period remained practically unchanged; for the samples with combined reinforced, the opening was 0.01 mm, and for the samples reinforced only with the woven meshes, it was 0.005 mm. The growth in the samples' deformations occurred basically owing to the formation and appearance of new cracks. The difference, detected at the outset of the crack formation, in the stiffnesses and saggings of the samples with the combined reinforcement and the samples reinforced only with woven meshes was maintained practically unchanged up to the completion of the tests (in the course of 2 months). As a result, the amount of the final sagging (after a 2-month holding, when the intensity of the saggings' increase was equalized for both types of samples) for the samples with the combined reinforcement exceeded by about 1.5 times the value of the final sagging of the samples reinforced only with the woven meshes. The ratio of the final buckling to the in- /43
itial sag for the specimens reinforced only by woven (wire) meshes equals about 2, and for the samples with the combined reinforcement, it equals about 3. We can assume that at an increase in the prolonged effective load to values adequate for the development (in the samples reinforced only with the woven meshes) of cracks with a size of 0.01 mm, the final bucklings and the strain moduli based on the bucklings, under the condition of the equality of K_{Π} and μ for both types of samples become equalized, although the nature of their deformation in the initial period of the holding time (at constantly effective load) will be analogous to that described above.

Section 7. Strength and Deformability of Reinforced Concrete During Shear

Depending on the system of applying the forces to the part tested, we differentiate two variants of shear stress:

1. The shear in the plane of a plate, as a result of which the tangential stresses originate, acting in the mutually perpendicular planes, normal to the shear plane (Fig.12,a).

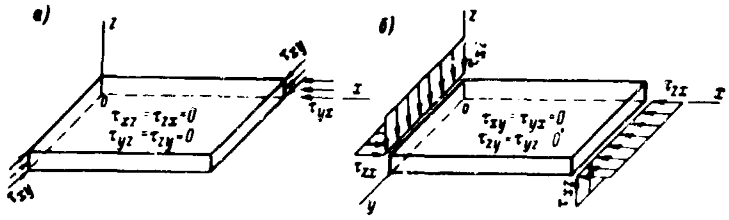


Fig.12. Diagram of Effect of Tangential Stresses During Shear: a - in the plate's plane; and b - in a perpendicular plane.

2. Shear in the planes perpendicular to the plate's plane (cut). As a result of such a shear, the tangential stresses originate, acting in the mutually perpendicular planes, one of which is parallel to the plate's plane; the second is arranged normally in relation to it (Fig.12,b). In the latter case, the shearing forces applied perpendicularly to the plate's plane cause in it, in addition to the deformations of pure shear, the stresses of crushing and bending, the effect of which distorts the test results. Therefore the operating capability of the reinforced concrete under shear was studied in the process of testing the flat plates for shear in their plane. At this time, we investigated the shear modulus in the plate's plane, the strength of plates during shear and the effect of various systems for /44
reinforcing the plates upon the shear modulus and the limit of crack stability.

We tested 4 series of plates, with 3 plates in each. The plates in all series had the identical form and measurements (Fig.13). The difference in the plates by series was caused by the application, for the reinforcement of the plates, both of woven meshes only, and of woven meshes in combination with an intermediate welded grating made of rod-type reinforcement with a diameter of 5 mm, and by a parallel and diagonal arrangement of the reinforcement relative to the plate's edges. The plates were made on a base of cement-sandy concrete of design grade 400. The characteristics of reinforcing the plates and the data for the verification of the strength of the concrete in the plates are presented in Table 9.

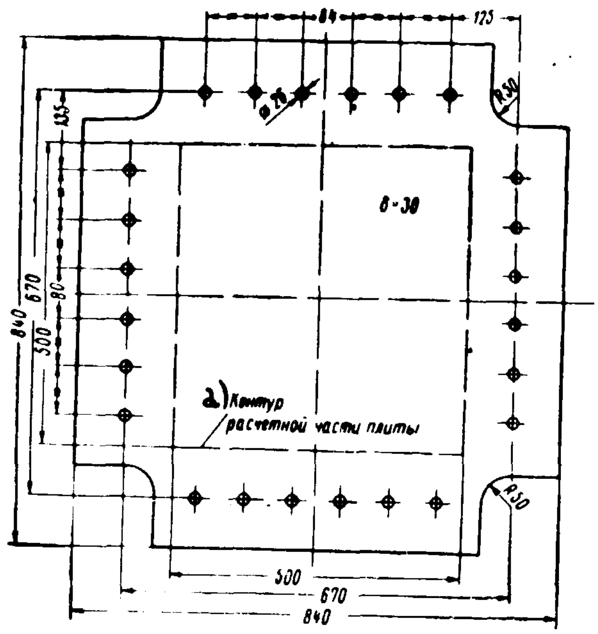


Fig. 13. Sample for Testing Reinforced Concrete for Shear in Plate's Plane.
Key: a) Outline of design part of plate

For reproducing the conditions of pure shear according to the stresses, the plates were tested in special devices, i.e. hinged four-linked devices. The plate which was being tested was fastened with bolts along the edges. During the expansion or compression of the four-link device along one of its diagonals, upon the edges of the plates, there acting relatively evenly distributed shearing forces.

Inasmuch as the purposes of the test included the construction of a stress- /45
strain diagram and a determination of the effect of the reinforcement systems on the strength and deformational properties of reinforced concrete during shear, the loadingo of the plates was conducted in steps at simultaneous recording of the loads and strains, and also of the appearance and width of the cracks' opening. The linear strains were measured along the expanding and compressing diagonals by mechanical comparators on a 500 mm base, and with wire-type resistance sensors. The deformations measured permit us to compute the shearing angle:

$$\gamma = \frac{\Delta l_p - \Delta l_c}{l}, \qquad (1)$$

Table 9

Results of Testing Reinforced Concrete Plates for [illegible]

Index of series	Characteristics of reinforcement: Type of reinforcement and orientation of installation	In direction of extending diagonal: K, cm^2/cm^2	In direction of extending diagonal: %	[illegible]	Tangential stresses: [illegible]	Tangential stresses: corresponding to [illegible]	Tangential stresses: γ_2 values [illegible]	[illegible] Modulus: in range of stresses $(0-\tau_1)$, $G \cdot 10^{-3}$	[illegible] Modulus: in range [illegible]
1C	12 mesh No.8 ∅ 0.7 parallely to outline of plate	1.[illegible]	2.[illegible]	[illegible] / —	47.3	[illegible].7	1[illegible]9	52.5	10.4
2C	[illegible] mesh No.8 ∅ 0.7 welded mesh of rods with ∅ of 5mm parallely to outline	0,[illegible]	[illegible]	[illegible] / [illegible]1	[illegible]	[illegible].5	[illegible]	[illegible].4	11,[illegible]
3C	12 mesh No.8 ∅ 0.7 mm diagonally to outline of plate	1,[illegible]	1,[illegible]	[illegible] / [illegible]	[illegible].5	[illegible]	[illegible]	[illegible].0	[illegible]
4C	[illegible] mesh No. 8 ∅ 0.7 mm and welded mesh of rods of ∅ 5 mm diagonally to outline	0,[illegible]	1,[illegible]	[illegible] / [illegible]	[illegible]	7[illegible]	[illegible]	24.0	[illegible]

where Δl_p, Δl_c - the values of variation in base of measurement of the expanding and compressing diagonals of plate, respectively; and l - the base of measurements.

Knowing the shear angle γ and the force applied to the four-link arrangement, we can calculate the shear modulus: /46

$$G = \tau/\gamma. \qquad (2)$$

The tangential stresses can be computed with accuracy, adequate for the practice, from the formula:

$$\tau = \frac{P}{a\delta} = \frac{P_Д}{F_Д}, \qquad (3)$$

where P - the force acting on the driving link of the four-link device; $P_Д$ - the force expanding the four-link device along the diagonal and established (registered) by the force met r of the testing machine; a - length of plate's side; δ - plate's thickness; and $F_Д$ - the area of the plate's diagonal section.

Table 10

Characteristics of Reinforced Concrete and Ferroconcrete Plates During the Impact Tests

Index of sample	Type of material	Strength of concrete R_{28}, kg/cm²	Plate thickness, cm	Number of meshes	Diameter (mm) of working reinforcement	Spacing of working reinforcement, mm	Use of steel per sq m of plate, kg
АЦ-3	Reinforced concrete	420	2,5	3	0,8	12	3,78
АЦ-5	"	400	2,5	5	0,8	12	5,40
АЦ-8	"	400	2,5	8	0,8	12	7,86
ЖБ-2,5	Reinforced sandy concrete	420	2,5	2	6	81	9,76
ЖБ-3	Ferroconcrete	470	3	2	4	50	5,38
ЖБ-5	"	405	5	2	6	84	9,76

Table 11

Data on Impact Strength of Reinforced Concrete and Ferroconcrete Plates

Index of sample	Value of Σohn in moment of appearance of first cleavages of concrete, kg/cm	In percentages of $\Sigma y\partial n$ for ferroconcrete plate 5 cm thick
АЦ-3	8 750	53
АЦ-5	15 000	94
АЦ-8	18 000	109
ЖБ-2,5	6 750	41
ЖБ-3	7 750	47
ЖБ-5	16 500	100

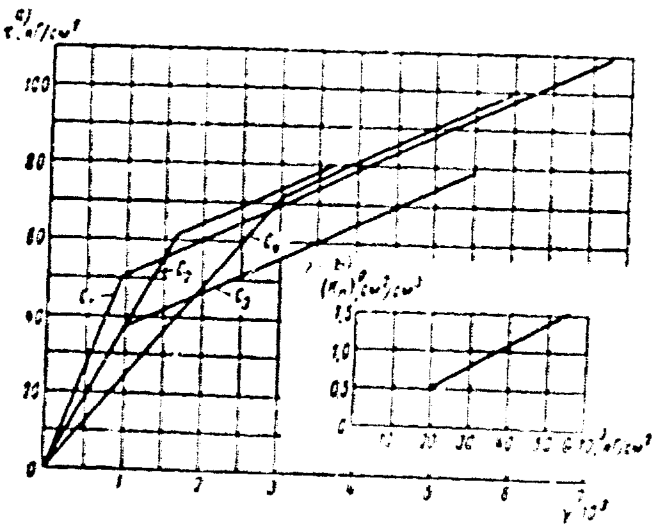


Fig. 14. Shear Diagram for Reinforced Concrete: a- relationship $\tau - \gamma$; b- dependence of initial shear modulus of reinforced concrete on specific reinforcement surface along the expanding diagonal of sample $(K_n)^p$ Key: kg/cm²

The results of testing the reinforced concrete for shear in the plate's plane were represented in the form of a functional relationship of the stresses and strains in a system of rectangular coordinates, where on the y-axis we have plotted the tangential stresses τ, while on the x-axis, we have shown the shear angle, γ.

In Fig. 14, a, we have shown a shear diagram for reinforced concrete. The shear diagram is depicted by the segments of straight lines, drawn at a certain angle to each other. The shear modulus was calculated from Eq. (2) separately for each sector of the diagram.

The rupture of the reinforced concrete plates during shear took place by way of the formation and subsequent opening of cracks, perpendicular to the expanding diagonal. The tests conducted demonstrated that during shear, reinforced concrete behaves as an anisotropic material. The resemblance consists in the fact that to the pure shear based on the strains, the pure shear based on stresses does not correspond. "Deformation anisotropy" is caused by the fact that in the plate structural changes occur, heterogeneous in various directions: as a result of the fact that the strength limit of reinforced concrete during stretching (expansion) is considerably lower than its strength limit during compression, a breakdown of the continuity of the concrete takes place along the sections, perpendicular to the expanding diagonal; however, no cracks form perpendicular to the compressing (contracting) diagonal. Since the direction of crack formation is not prescribed in advance, but is determined entirely by the deformation per se, an apparent anisotropy develops. In this connection, it is postulated that during the entire loading process, the principal strain axes maintain their direction. If at some stage of the load, the principal axes alter their direction considerably as compared with the previous stages, the deformation-type anistropy in the previous stages of load in relation

to the subsequent stages will no longer be the apparent, but will constitute the actual anistropy.

In Table 9, we have presented the results of the tests conducted on reinforced-concrete plates having functioned under conditions of pure shear. The arithmetic means of the appropriate values have been computed based on the results of tests made on three plates. The data in Table 9 provide evidence that the initial shear modulus of reinforced concrete is connected linearly with the value of the specific surface of the mesh wires, oriented in the direction of the expanding diagonal of the plate (Fig. 14, b). As concerns the values of the shear modulus in the vector after the truncation of the diagram, for all the plates, it proved to be practically the same.

The utilization of plates for reinforcement along with the fiber meshes of an intermediate welded grating made of rods 5 mm in diameter reduces the value of the tangential stresses τ_1^{TP}, corresponding to the width of cracks' opening 0.005-0.01 mm, as a rule developing in the sections where the transverse rod-type reinforcemnt was located. It should be commented that the appearance of individual cracks of the indicated opening width does not immediately cause a change in the nature of deformation, i.e. the value of the shear modulus does not change at once after the appearance of the first visible cracks. /4
The truncation of the diagram takes place at the strain τ_1, to which there corresponds not the individual, but the fairly thickly arranged visible cracks, in which the opening width remained practically constant within the limits of 0.005-0.01 mm.

After the truncation of the diagrams, the increment in the deformations in the direction of the expanding diagonal takes place chiefly owing to the increase in the width of the opening of the cracks which have formed. The values presented in Table 9 for the tangential stresses τ_2 correspond to the width of

cracks' opening of the order of 0.1 mm, i.e. to the moment when we have a disruption in the nature of the stable deformation of the plates.

A comparison of the values of the initial modulus and of the critical resistance of reinforced concrete to shear during parallel and diagonal systems of plates' reinforcement permits us to note that the values of G_1 and τ_1 for the parallel reinforcing system proved to be the highest. This is explained by the fact that in the parallel system of reinforcement, the value of the specific surface of the meshes and of the reinforcement factor in the direction of the expanding diagonal is higher than the appropriate values under the diagonal system of reinforcement.

Section 8. Functioning Capability of Reinforced Concrete During Impact

In the application of reinforced concrete, as of ferroconcrete in general, in the capacity of a hull material for floating facilities, the maximum interest is represented by the local disruptions from the direct effect of impact: the puncturing of plates, the breakage of the watertight sheathing under a blow, etc.

It is known from the practice that the local breakdowns of a ferroconcrete plate under impact depend on the force of the blow and the plate thickness. In this connection, on the plate's surface from the side of the blow, there will appear concentric and radial cracks, while on the opposite side, we will find cracks, and then cleavages of concrete. Evaluating the effects of the blow, we proceed from the physical nature of the processes transpiring under impact and the features of the mechanical properties of the design material.

The general theory of the design for impact loads has not yet reached the level when the strength criteria can be clearly formulated, based on the

analytical relationships between the strains developing in the material under impact, and the deformations. The special studies devoted directly to the distribution of stresses and strains during the puncturing of a ferroconcrete plate of limited thickness are completely lacking. Therefore, many private theories of impact exist. Several of them shed light on the physical processes occurring during the breakage of a ferroconcrete plate from the local effect of a blow.

/49

In particular, according to the theories assuming for a basis the contact phenomena on the contiguity surface of the colliding bodies, outside of and on the boundary of the pressure line, the case of pure shear takes place. Therefore, if we take into consideration that the breakdown of concrete and of materials similar to it under conditions of pure shear based on stresses takes place from the overcoming of the strength limit of the material under expansion (see Section 7), it turns out that these zones also comprise the most risky ones in the sense of the disruption of the material's continuity. In addition, the value of the shear stresses (and by the same token, the time of appearance of cracks in the materials similar to concrete) is influenced by the form of the surfaces of the colliding bodies. The greater the radius of the surface of the body receiving the shock, the higher the value of the shear stresses which are appearing.

Another unique form of the breakdown of ferroconcrete plates, i.e. the cleavages (splitting off) is associated with the propagation of the longitudinal waves, caused by the impact. Under the effect of a blow onto a ferroconcrete plate, a compression wave propagates inward from the surface at the point of impact. During the approach to the free surface, it is reflected with the origination of the expansion wave. At the moment when the incident and reflected waves do not overlap, and the stresses in the reflected wave reach the

strength limit of the concrete during expansion, the cleavage of the concrete takes place from the side opposite the impact. The physical aspect of the cleavage phenomena is particularly complicated when the plate through its depth consists of several layers. In this case, in the interface of the various materials, three waves interact: the incident, the reflected and the forward.

In this manner, the cleavage phenomena comprises a complex aggregation of breakdowns, caused by the transformation of the compression energy into the energy of expansion with allowance for the features of the physical-mechanical properties of concrete, specifically: of the abruptly varying resistivity during the deformations of contraction and expansion.

Thus, the existing attempts at a theoretical review of the occurrences accompanying the local effect of impact testified to the direct connection of the impact resistance of the ferroconcrete plating with the ability of the concrete to resist expansion and pure shear.

Among the dynamic studies, not intended to establish a relationship between the stresses and strains originating in the material under impact, the purpose of which is the determination of the relative advantages of the various materials or of the different types of the same material, we include the tests of plates with a falling weight. In conformity with the delineated problems, the existing diversity in the procedures for conducting such tests precludes the merited comparison of the results of the tests conducted by various researchers. In our view, the most graphic results were obtained by A. A. /50
Kudryavtsev* in a comparison of the impact strength of reinforced concrete and of ferroconcrete plates. For the comparison standard, he adopted a ferroconcrete plate 5 cm thick.

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*A. A. Kudryavtsev. "On the Question of Impact Strength of Reinforced Concrete & Thin Ferroconcrete Plates." In the collec: "Reinforced Concrete & Reinforced Concrete Designs", Editorial Board of the Journal "Bulletin of Technical Information" Glavleningradstroy, 1959.

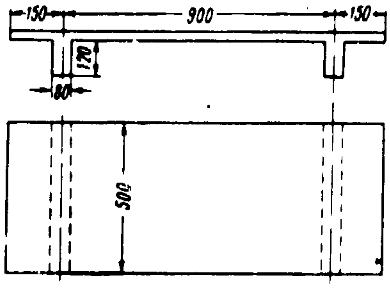

Fig. 15. Design of Reinforced Concrete Plates During Impact Tests.

The ferroconcrete i reinforced concrete plates had stiffening ribs (edges) which served as support for the plate section subjected to impact. The dimensions of these sections in plan is 90 X 50 cm (Fig. 15). The basic characteristics of the reinforced concrete and ferroconcrete plates are presented in Table 10. The impact testing of the plates was conducted on a special stand by dropping a 25 kg weight onto the plate. The form of the impacting surface of the weight was spherical with a sphere radius r = 25 cm. In the testing process, we observed the width of the cracks' opening. The evaluation of the plates' strength was conducted according to the provisional standard of impact strength ΣpHn, equalling the sum of products of the weight p times the height of fall H, and times the number of impacts n from a given height of fall. The impacts were conducted prior to the development of cleavages, which were adopted for the breakdown of the plates.

The dependence of the opening of cracks in the plates on the value of the arbitrary standard of impact strength ΣpHn prior to the time of appearance of chipping away of concrete (Fig. 16) indicates that the reinforced concrete plates, as compared with the usual ferroconcrete plates, have an increased resistance to the impact loads.

The combined data concerning the impact strength of reinforced concrete and ferroconcrete plates are presented in Table 11.

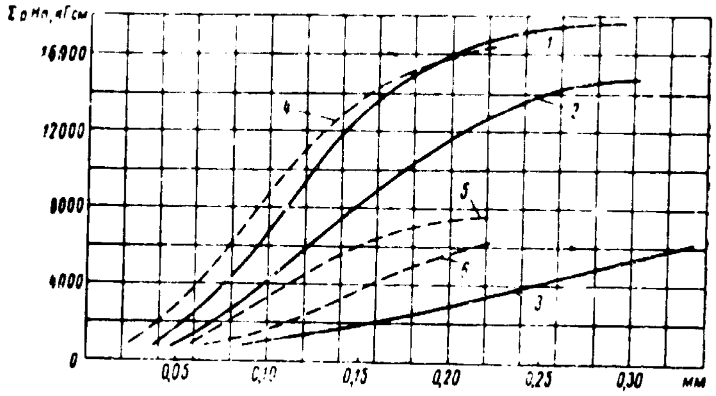

Fig. 16. Dependence of the Opening of Cracks in Reinforced Concrete and Ferroconcrete Plates Upon the Value ΣpHn During the Impact Tests. Key: a) kg/cm, 1, 2, 3- reinforced concrete plates A4-3, A4-5; A4-8; 4, 5, 6- ferroconcrete plates with a thickness 5.0, 3.0 and 2.5 cm.

In the experiments of the authors on the study of the impact strength of reinforced concrete and ferroconcrete plates conducted by a similar procedure (with the sole difference that the weight of the falling load p = 10 kg, /52 while the radius of the impacting sphere r = 15 cm), it was also established that a concrete reinforced plate 2.5 cm thick, reinforced with 6 no. 10 meshes of wire 1.0 mm in diameter and with an intermediate welded mesh of rods 5 mm in diameter has the same impact strength in respect to the cleavages of concrete as the ferroconcrete plate 5 cm thick, reinforced with two rod-type meshes, one of which is formed from rods 8 mm in diameter with a spacing of 12.5 cm, while the other is made of rods 6 mm in diameter with a spacing of 10 cm.

As concerns the quantitative indexes of the resistance of plates to puncturing during impact in the tests of such a type, they depend upon many factors, including the weight of the falling object, the rate of the impact, the geometry of the colliding bodies, etc. Therefore, the quantitative results of the tests of plates by the falling weight, conducted under specific conditions, can not be extended to plates with other dimensions or to the case of other test conditions.

The only results having importance which can be obtained from these experiments consist in a comparison of the nature of the breakdown of the reinforced concrete and of the ferroconcrete plates, in the qualitative evaluation of the influence of the systems for reinforcing the plates. At this level, the available experimental data provide the possibility of concluding that the dispersed reinforcemnt promotes to a considerable extent an increase in the impact strength. As concerns the kinetics of the cleavages occurring during impact, larger cleavages take place in the ferroconcrete plates. In the reinforced concrete plates, up to the start of the cleavages, we do not find disruptions in the continuity of the reinforcement, or discontinuities in the thin wires of the meshes, but the crushed concrete is held back by the meshes.

The referenced feature of the breakdown of reinforced concrete plates during impact as compared with the ferroconcrete plates is significant in the utilization of reinforced concrete for the plates of marine (ship) sheathing, since in this case, the seepage of water through the damaged places is slight as compared with the through hulls, and the damages can be repaired fairly simply.

Section 9. Watertightness of Reinforced Concrete

The investigation of the watertightness of reinforced concrete was conducted in plates existing in a stress state under the effect of hydrostatic pressure. In this connection, the influence of the hydrostatic pressure as a constant uniformly distributed load on the plate was prolonged (the holding time was 40 days).

The tests were conducted on special hydraulic stands, on the upper open part of which we fastened the sample being tested (Fig. 17). At the top of the plate, parallel to its long side, we installed an intermediate support, which divided the plate into two equal parts measuring 850 X 315 mm. /53

In the process of testing the reinforced concrete plates, we measured the bucklings (sags) in the center and quadrants of the plates' span and the external load, and we also observed the nature of the crack formation, with a measurement of the width of cracks' opening. The required pressure value was developed by changing the water column by way of moving along a vertical stand a movable tank with a water-measuring glass, and monitoring was also achieved by observing the sample manometers, installed on each stand.

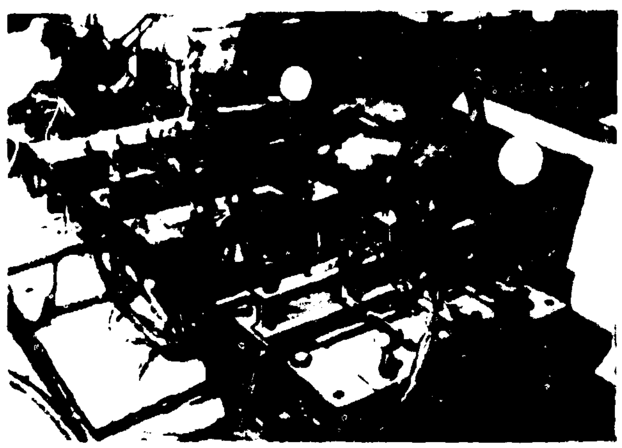

Fig. 17. Equipment for the Prolonged Testings of Reinforced Concrete Plates by Hydrostatic Pressure.

The loading of the plates was conducted according to the graph (Fig. 18).

All the plates had the same thickness, 20 mm and were reinforced only by the woven meshes No. 5, 8 and 10 of 8, 12, and 8 layers each respectively through the entire depth of the sample. The specifications of reinforcing the plates were as follows:

Mesh No. 5 ....K_n = 3.08 cm^2/cm^3 μ = 2.79%

Mesh No. 8...........K_n = 3.04 cm^2/cm^3 μ = 2.64%

Mesh No. 10..........K_n = 2.28 cm^2/cm^3 μ = 2.86%

The strength of the concrete to compression at the time of installing the plates for the tests, established by the testing of control blocks 7 X 7 X 7 cm comprised 600-700 kg/cm^2.

In an examination of the plates prior to raising the pressure, we detected at the corners and along the edge of the clamping frame the cracks with an opening of 0.05 mm. These cracks were caused by the squeezing of the plates' edge during the tightening of the clamping bolts. /54

At an increase in the pressure to 0.6 kg/cm^2, we noted the condensation of moisture on the plates along the edge of finishing and under the intermediate support.

At a pressure of 0.8 kg/cm^2, when the amount of stresses in the expanded zone of the section under the intermediate support, determined with the formula $\sigma = M/W$, equalled 90 kg/cm^2, the plates were left under the load.

After a short holding under the pressure of 0.8 kg/cm^2, on the surface of the plate in the places where we previously noted the condensation, water drops appeared. In the course of several days, the intensity of filtration remained without change, and then it began to decrease and finally toward the end of the second week of keeping the plates under pressure, the filtration (seepage) of water stopped. This can be explained by the fact of the "self-compaction" of cracks, i.e. by the crystal formation in the cracks on a base of the calcium carbonate eroded from the concrete. In the lesser stressed sectors of the plate between the supports, where under the effect of a prolonged hydrostatic pressure of water, the opening of the cracks on the expanded surface of the plates comprised not less than 0.005 mm, no indications of water seepage appeared.

The nature of the increment of the bucklings of plates in the process of keeping them for 40 days under the effect of constant hydrostatic pressure is similar to that described in section 5 for the samples reinforced only by the woven grids (fiber meshes). The difference consists in the fact that under the conditions of constant contact with water, the stabilization of the amount of sags takes place in a shorter time (during a half month).

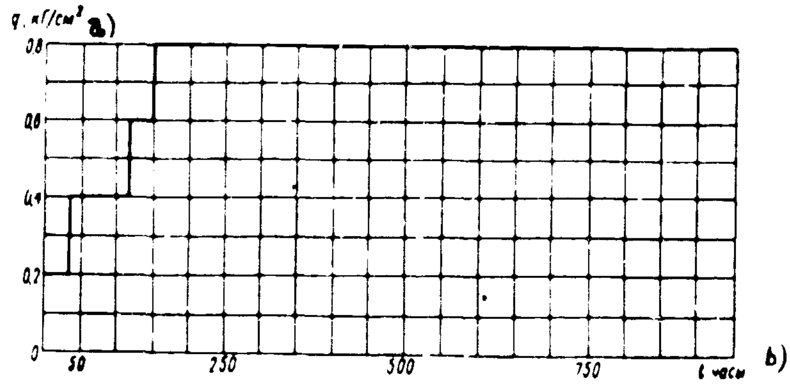

Fig. 18. Graph Showing the Increase in the Hydrostatic Pressure. Key: a) q, kg/cm^2; b) t, hours.

The tests conducted indicate that from the viewpoint of watertightness, of the plating of a hull made of reinforced concrete, the cracks with an opening size of 0.05 mm do not present any danger. However, from the standpoint /55 of long life (preserving the reinforcing grids), the presence of cracks with an opening of 0.05 mm, under which in the initial period of effect of hydrostatic pressure, the dropwise filtration of water takes place, is not permissible.

These findings are confirmed by the data provided by other researchers, particularly the data obtained in the GIIVT. The GIIVT conducted a study of the watertightness of reinforced concrete in the flexured samples 1000 X 250 X 30 mm. From the side of the expanded zone of the samples, a watertight bell was tightly fastened into which water was fed under pressure from a tank.

The stressed state of the samples corresponded to the appearance on the expanded surface, of cracks with an opening of 0.01 mm. The pressure was increased in steps, with holding at each step for 3 hours. As a result, it was established that the appearance of signs of water seepage (darkening of the compressed zone) takes place at pressures of around 1 atm. The dropwise flow occurred at a pressure of 1.5 atm. The difference in the values of the pressures causing the indications of seepage in our samples and in the GIIVT experiments,

is explained by the varying stress-strain condition of the samples: in our tests, the extent of cracks' opening was 0.05 mm; in the GIIVT tests, it was 0.01 mm.

If we take into consideration that the draft of vessels made of reinforced concrete does not exceed 5 m, i.e. the maximal pressure of water on a plate in the hull sheathing comprises not over 0.5 atm, we can consider that the full watertightness of the ship hulls made of reinforced concrete, even in the presence of surface cracks on the plates with an opening of up 0.01 mm, will be assured.

However, taking into account the slight thickness of the protective layer of concrete and the intensively developed surface of the reinforcement of woven meshes in the reinforced concrete, for the surfaces contacting the water, we should recommend the application of protective coatings, presenting the penetration of moisture into the reinforced concrete plates, and thereby protecting the woven screens from corrosion.

Section 10. Resistance of Reinforced Concrete to Freezing

The resistance of reinforced concrete to freezing was studied by comparing the elastic-strength characteristics obtained during the testing of control samples and of samples having passed through 150 cycles of alternate freezing and thawing, for pure bending and axial expansion. The control samples were kept under normal moisture conditions.

The reinforced samples corresponding to the shipbuilding reinforced concrete were subjected to the tests.

Prior to testing, the samples were inspected, weighed and placed on it, 4 pieces each in special boxes. The boxes were placed in a ship, which was filled with water at a temperature of +15°C. The submergence of the samples was achieved to 1/3 of their height. In such a condition, the samples were

kept for 12 hours, whereupon water was added to the ship to the level corresponding to two-thirds of the sample's height, and the samples were again kept for 12 hours. After the elapsing of 12 hours, the ship containing the samples was filled with water up to the complete submergence of the samples (1-2 cm above their upper edge). The samples were kept in such a state for 24 hours, whereupon they were again weighed to determine the percentage of water saturation. Then the samples in the boxes were placed in a refrigeration room and were frozen for 5 hours at a temperature of -17°C. The frozen samples were thawed for 6 hours in water at a temperature of +15°C. After each cycle of freezing and thawing, the samples were inspected.

The samples having passed through 150 cycles of freezing and thawing were tested for axial expansion and pure bending simultaneously with the control samples.

It was established as a result of the experiments conducted that after 150 cycles of alternate freezing and thawing, the reinforced concrete samples had the same values for the elastic-strength characteristics as the control samples, having been kept under normal moisture conditions.

Chapter II. DESIGN OF SHIPS MADE OF REINFORCED CONCRETE

Section 11. Most Typical Examples of General Composition of Reinforced-Concrete Hulls /57

A considerable influence upon the hull design of the planned and built reinforced concrete ships is exerted by the experience of the steel and ferroconcrete shipbuilding. In the planning of the reinforced concrete hulls, the greatest popularity has been gained by the chiefly mixed system of assembly.

The general requirements for the reinforced concrete hulls of ships remain about the same as for the ship hulls made of other materials. The main ones are: operational adaptability, reliability, technological effectiveness, and economy of design at minimal weight. However, the design embodiment of these requirements is affected by the specific features of reinforced concrete as of a shipbuilding material in general, and as of a variety of ferroconcrete in particular.

Thus, as compared with ordinary ferroconcrete, reinforced concrete is most suitable for the production of designs of the shell type. The application of reinforced concrete in place of ferroconcrete introduces a significant simplification into the technology of producting the shell designs, specifically it assures the possibility of the formless preparation of the individual designs and hulls of ships as a whole. In this manner, preserving all the advantages of ferroconcrete during its functioning in the composition of the shell-type designs, at the same time the reinforced concrete permits us to simplify considerably the technology of manufacturing the shell designs, and hence to reduce their unwieldiness and cost. However, if we are oriented only on the completeness on the sectional method of the construction of reinforced concrete ships, the application of the shell-type form of hull proves to be inefficient, since the curvilinear

outlines of the hull lead to an appreciable (as compared with the flat) complication and increased cost of the technological equipment, and to an increase in the labor and cost of producing the prefabricated elements and the formation of the hull on the building slips. /58

From what has been said, we can conclude that one of the determinative conditions of the choice of a shell design of a reinforced concrete hull is the method of construction.

It is known that the general tendency in ferroconcrete shipbuilding consists in the transition to the completely prefabricated construction method, guaranteeing (with the simplified flat outlines of the hull) the industrial production of the prefabricated parts of the hulls and ships superstructures. This tendency has exerted an influence also on the development of the hull design in reinforced concrete ships. The planners, not taking into account the specifics of reinforced concrete, have not utilized the possibility of the formless preparation of the hull by the monolithic method, and has suggeste as a basis, the sectional plane design of hulls.

Let us consider the essential design systems of reinforced concrete marine hulls based on the results of the planning design studies, and also of ships already built.

Reinforced Concrete Ships with Hulls Made in the Form of Circular Unframed Cylindrical Shells. An example of such hulls is represented by the planning studies of the barge-platforms with a hoisting capacity of 600 tons (Fig. 19) and of a platform with a length of 20 m (Fig. 20). For the indicated hulls, it is typical that in them the basic support element is the reinforced concrete shell, not supported by beams of the set. The strength and stability of the reinforced concrete shells is provided by their spatial-curvilinear form.

The reinforced concrete circular cylindrical shells are interconnected

into a unified hull by transverse bulkheads and by a deck also made of reinforced concrete. The hull shells of the barge-platform have along the bottom and on the deck supports in a longitudinal direction in the form of reinforcing rods of large diameter for receiving the forces from the overall longitudinal bending.

If we analyze, even if only in general outlines, the operating and technological qualities of the reinforced concrete hulls made in the form of circular cylindrical shells, we can notice that:

- the hulls have a small coefficient of completeness, and hence a high draft and a large wetting surface as compared with the flat-deck hulls;

- the conditions for utilizing the hatches in the hulls under the various rooms and for the placing of cargos in them prove to be less favorable than in the flat-deck hulls; and

- the construction of hulls both by the prefabricated and by the monolithic method in the absence of mechanized production of the circular cylinders is quite complex.

At the same time, a circular cylindrical shell is the most improved form from the viewpoint of utilizing the elastic-strength properties of reinforced concrete.

Therefore, in the case of the development of the technique of the mechanized production of circular reinforced concrete cylinders during their mass output, these cylinders can be utilized effectively for the formation from them of hulls of such floating facilities, on which the installation of the cargo and service quarters is done outside of the hull (barge platforms, pontoons etc.), and the travel speed is slow.

Reinforced Concrete Unformed Hulls in the Form of Simplified Shells. In their configuration, the lines of such ships are close to the lines of the monotypical metallic and plastic vessels. The basic support element of the hull is

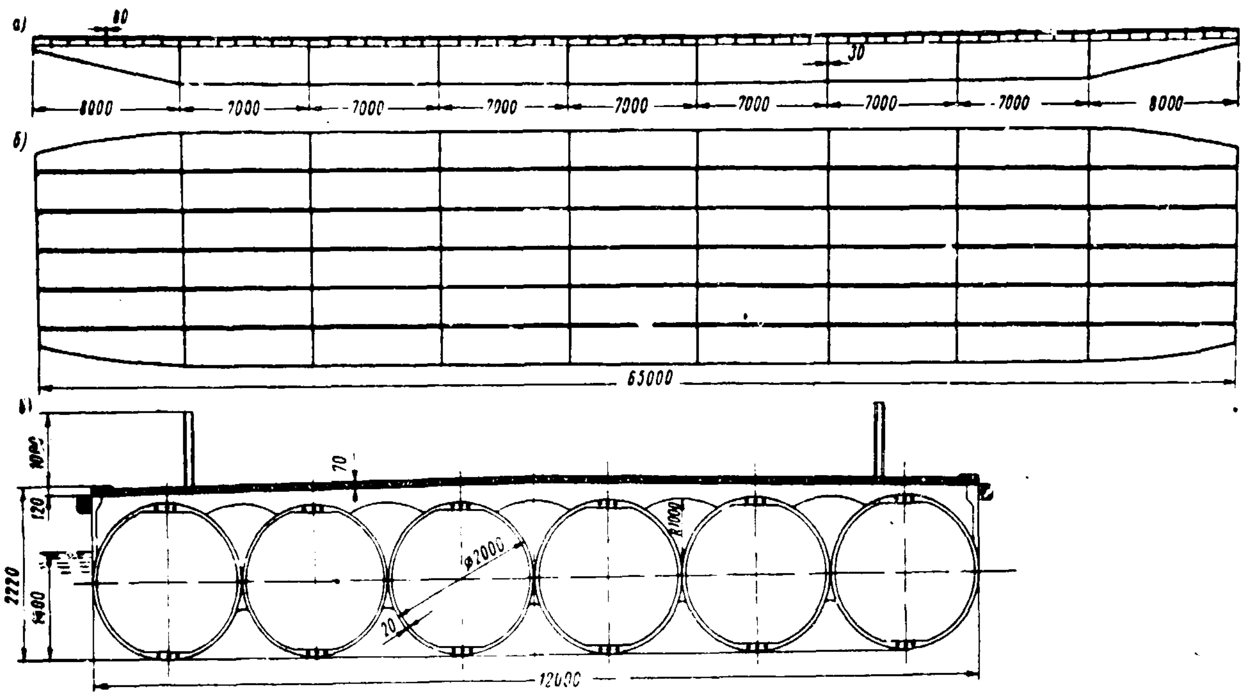

Fig. 19. Barge-Platform Made of Reinforced Concrete: a- section along DP; b- plan of barge with deck covering removed; c - transverse section. /60

the reinforced concrete shell, not supported by beams.

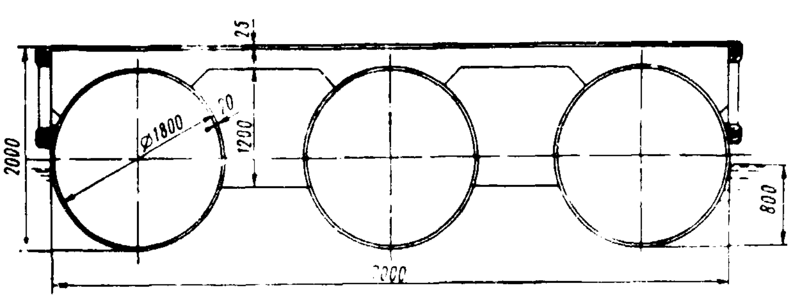

Fig. 20. Cross section of Reinforced concrete hull of a platform of shell-type design.

The conditions of the operational adaptability and hydromechanical qualities of the ship are easily assured under the combination of the curvilinear and flat surfaces of the hull. However, the unframed type of hulls can be made only for small ships of the lifeboat type, surface and excursion cutters, etc, experiencing

slight forces from the overall longitudinal bending. In Fig. 21, we have shown a general view of a pleasure craft with a hull in the form of a smooth reinforced concrete shell, prepared by the monolithic method with the application of formless concreting. Such ships are mass produced by the Windboats Company.

Specifications of Ships

Length, m 11.3
Width, m 3.08
Draft, m 0.75
Thickness of sheathing, mm 22

Quite significant is the fact that these ships combine most fully the design and engineering features of reinforced concrete (the shell-type design and the formless method of construction), providing thereby the possibility of /61
satisfying the conditions of operating adaptability and the possibility of deriving high hydromechanical qualities of the ship.

Reinforced Concrete Hulls with Ferroconcrete Assembly Beams. In their overall frames, these hulls do not differ basically from the steel and ferroconcrete ones.

In the framed design, the outer sheathing and all of the plates of the deck, bulkheads and partitions as a rule are made of reinforced concrete, while the framing is made of ferroconcrete or steel.

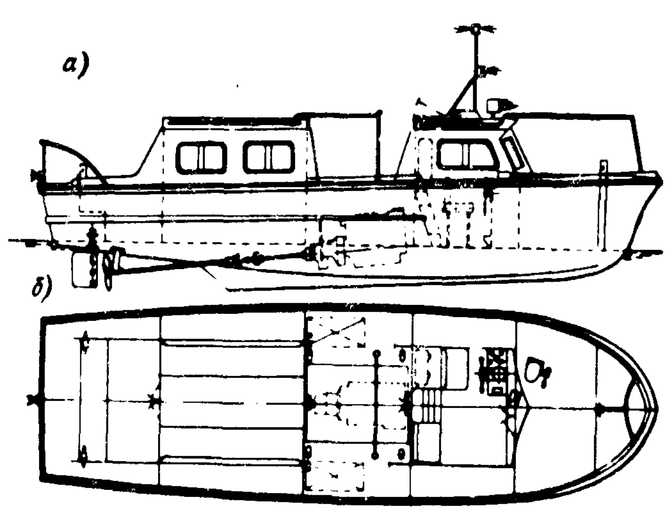

Fig. 21. Pleasure Launch made of Reinforced Concrete. a- longitudinal section; b- top view

The hulls of the planned or built framed reinforced concrete ships have a transverse or combined system of framing.

The outlines of the framed hulls can be the mold type or simplified, depending on the purpose of the ship and the method of its construction. In case of orientation on the fully framed method of construction, the most effective in respect to labor and cost of construction, especially on a large scale, are the simplified configurations (flat sides, bottom and transom ends). However, there are no bases for giving preference for the framing method of construction, if the simplification of the lines leads to a considerable depreciation of the hydromechanical qualities of the ship. For the construction of the frame-type hulls of mold (template) outlines, we can apply successfully the monolithic method, with the utilization of the formless concreting, which, not requiring high skill in the workers or expensive equipment, can easily be mastered by any shipyard. /62

Thus, in the template outlines of a hull, we should use the formless monolithic method, especially in the individual construction of ships (or in small batches).

In Fig. 22, we have shown a cargo ship of framing design, of template lines with a cargo capacity of 100-120 tons. The system of framing the hull along the bottom is longitudinal, and along the sides and deck it is transverse. With five transverse watertight bulkheads, the hull is divided into six compartments. The depth of the reinforced concrete plates of the sheathing is 25 mm. The framing beams (150 - 350) X (40 - 60) mm are made of ordinary ferroconcrete. The frame spacing is 700 mm, and the distance between the frames under the longitudinal system of framing the bottom equals 1400 mm.

An example of a ship with a hull and superstructure made of reinforced concrete, having simplified outlines in combination with the template ones, can be provided by the driftwood hoisting crane with a cargo capacity of 10 tons (Fig. 23).

Specifications of Floating Crane

Length, m	24
Width, m	10.4
Molded depth, m	2.2
Framing system of hull	transverse
Amount of spacing, mm	700
Thic ness of sheathing, mm	25

In the stern part of the hull, in the region of frames 19-33, we have a slot 3.8 m long, the continuation of which is provided by a tunnel passing into the bow counter. The bow and stern of a sled type are raised.

The hull has two longitudinal and seven transverse bulkheads. The longitudinal bulkheads are a continuation of the internal sides of the recess (slot).

The design thickness of the reinforced concrete sheathing of the bottom, sides and deck is 25 mm; that of the longitudinal and transverse bulkheads is 20 mm; and that of the transoms is 30 mm. The framing beams with a cross section (100-200) X (40-50) mm are made of ordinary ferroconcrete.

The method of building the hull and the superstructure is entirely prefabricated. In this operation, the individual sectional elements are ribbed designs, i.e. reinforced concrete plates, fastened by the ferroconcrete framing beams, made on a base of cement-sandy concrete. The production of the prefabricated parts was conducted on the bottom plates with the ribs upward.

For reinforcing the sheathing plates of the hull, we used six pieces of woven mesh, No. 10 based on GOST 3826-47, an intermediate rod-type mesh with a reinforcement diameter of 4 mm according to GOST 3282-46. For reinforcing the framing beams, the reinforcement diameter was 6 and 10 mm based on GOST 380-60 and the reinforcement of periodic profile with diameter of 16 and 20 mm, grade 35 GS based on GOST 5059-57.

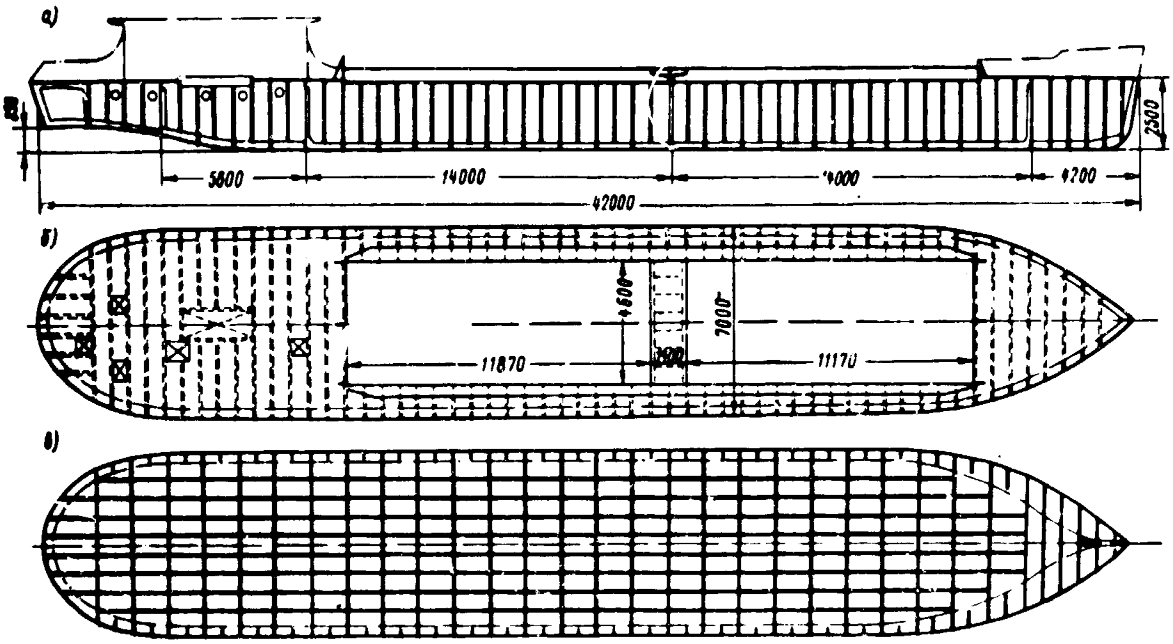

Fig. 22. Diagram of Hull Framing of Dry-cargo Steamer with a Cargo Capacity of 100-120 tons: a- section according to freight agreement; b - layout of deck; and c - layout of bow. /63

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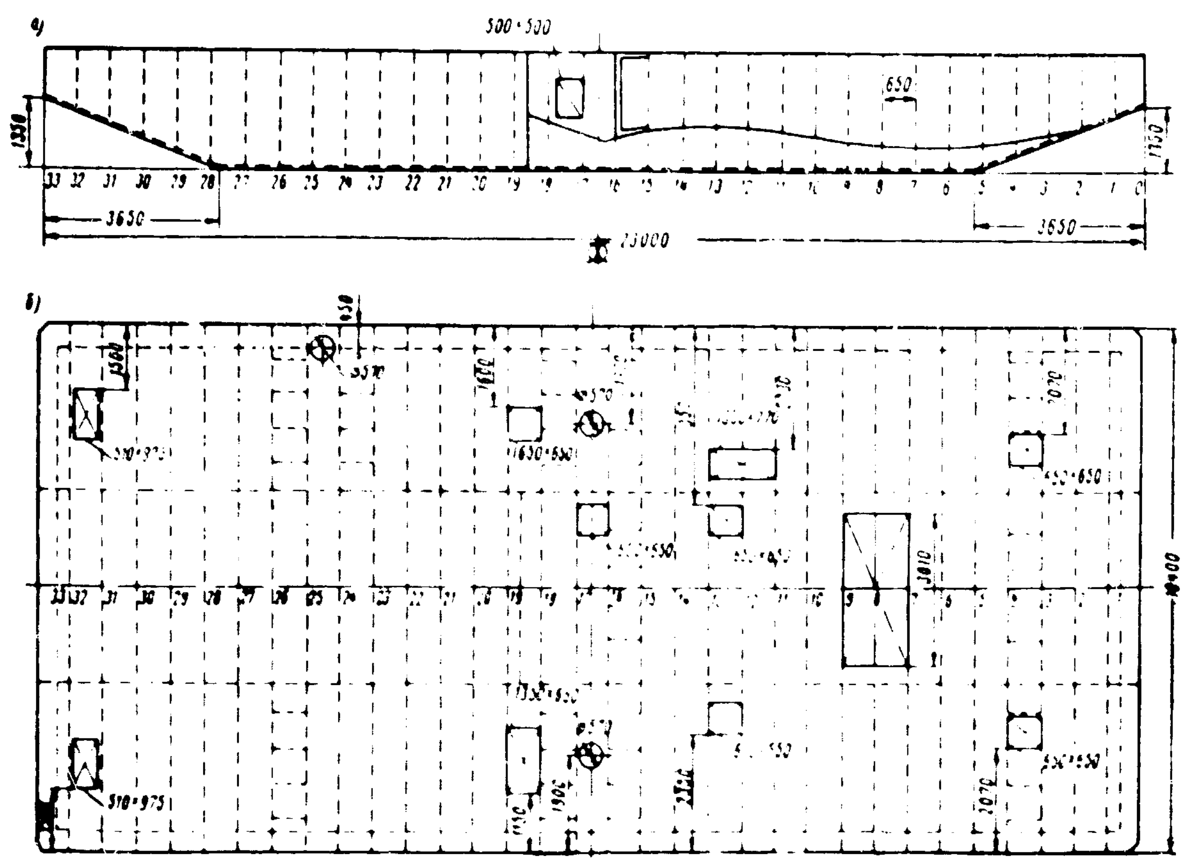

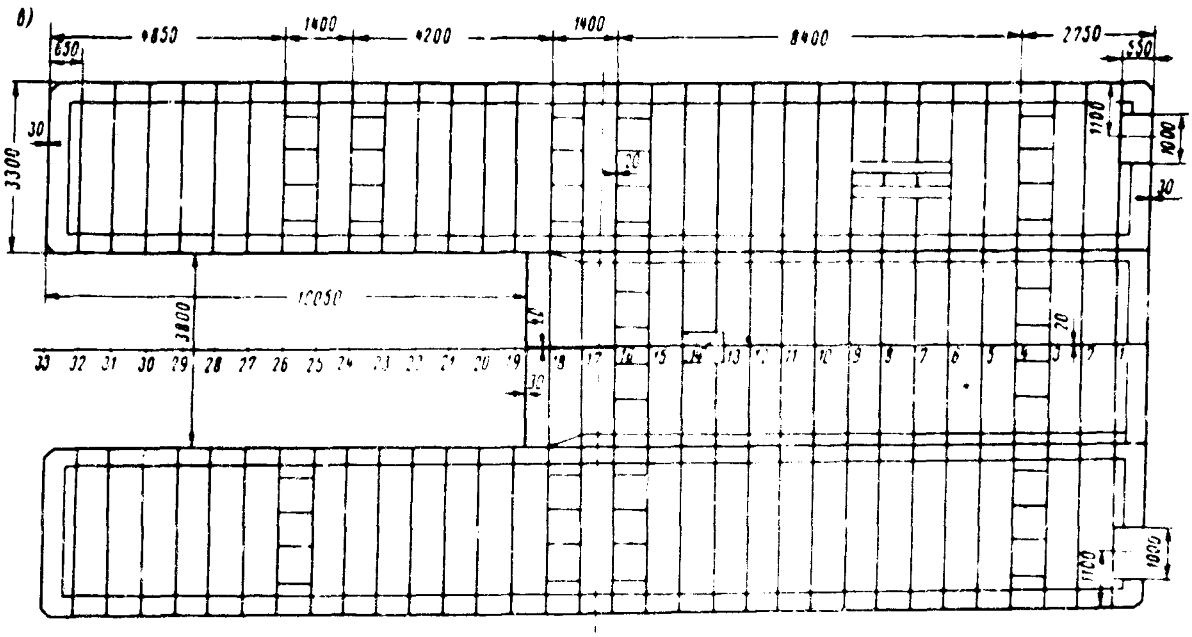

Fig. 23. Design Drawing of a Hull of a Floating Crane: a - section according to freight agreement; b - layout of deck; c - layout of bow

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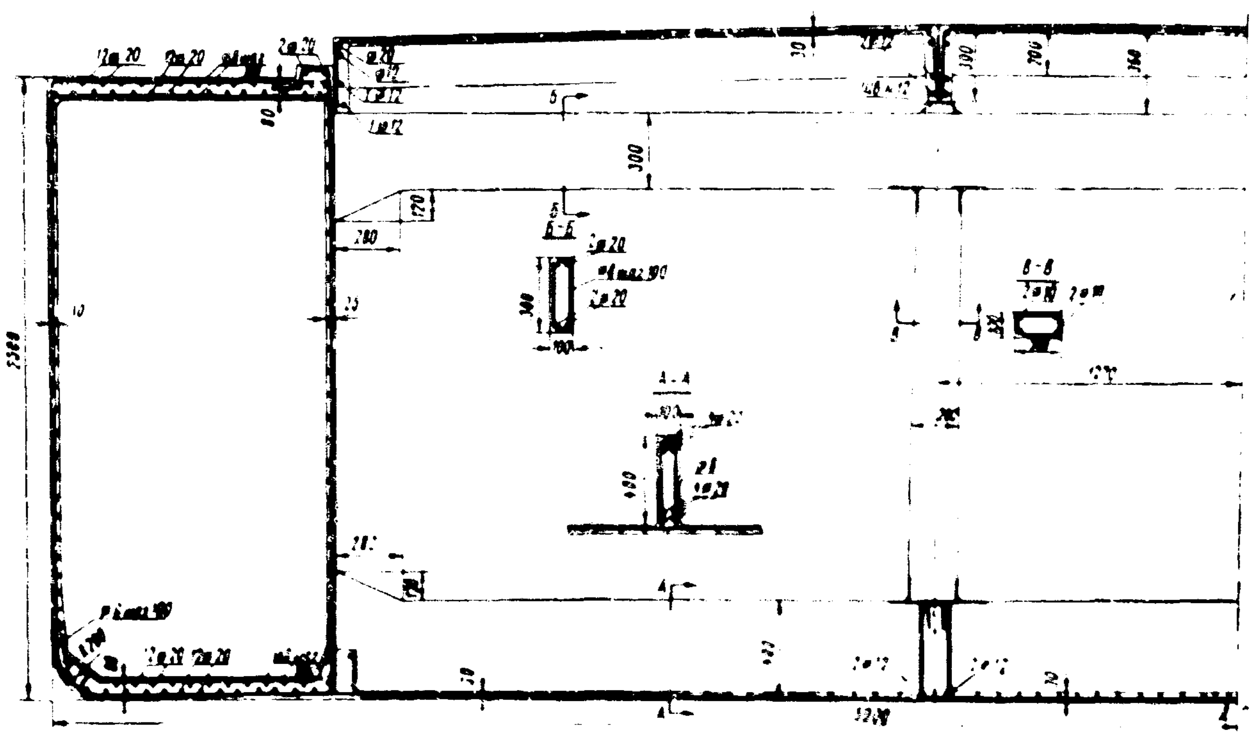

Fig. 24. The Midship Frame of a Barge Platform with a Cargo Capacity of 6?0 Tons

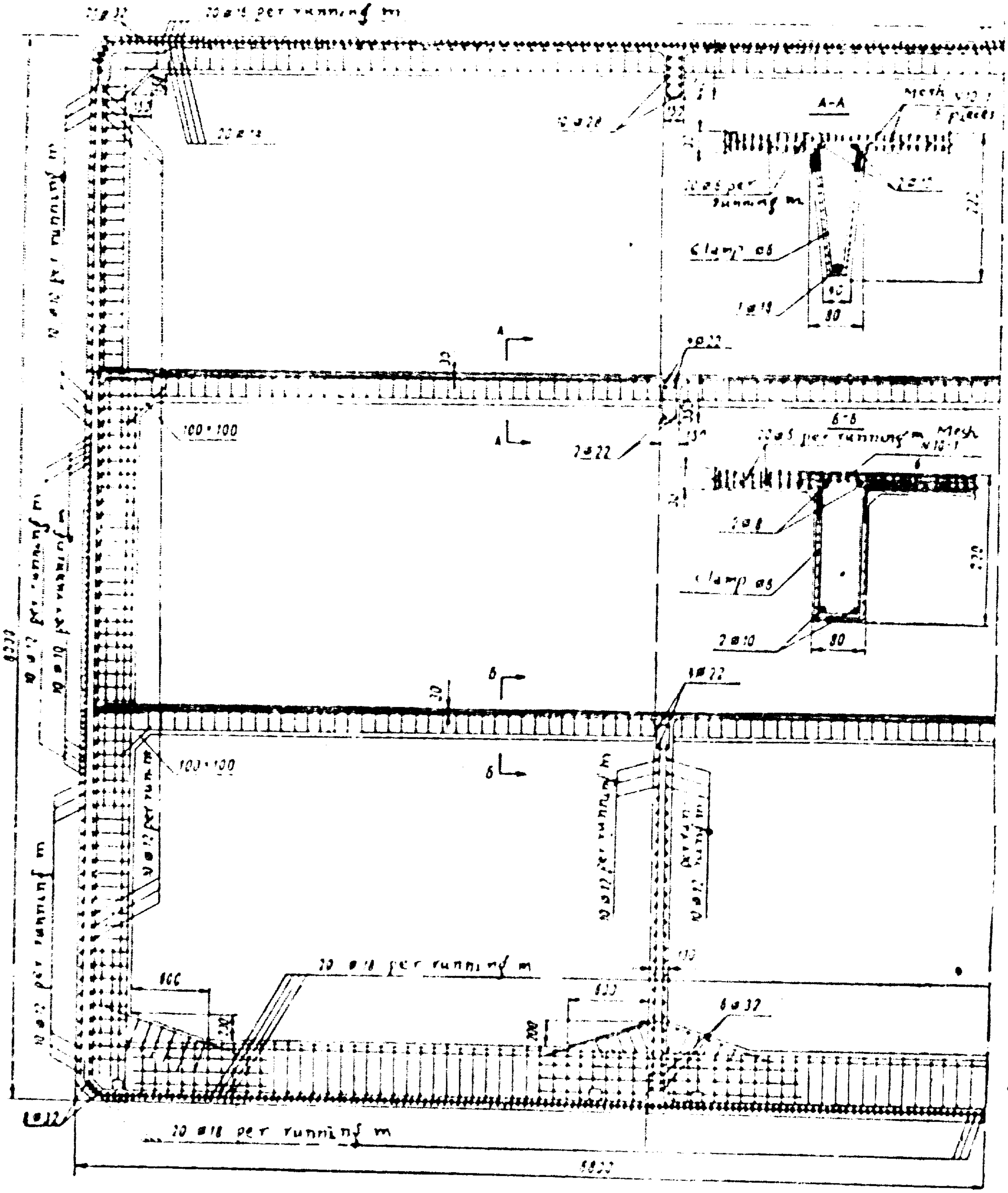

Fig. 25. Cross section of hull of marine vessel with lower and middle decks made of reinforced concrete.

In all of the examples considered of the designs of the reinforced concrete hulls, the reinforced concrete sheathing of the hull, has been assigned from the condition of its participation in the overall longitudinal bending of the hull. However, we should note that for reasons of inadequate study of the properties of reinforced concrete in the period of planning the experimental ships. the participation of the reinforced concrete plating in the absorption of the forces from the overall bending of the hull was partly restricted by the designers. There occurred cases of the complete exclusion of reinforced concrete plating from the composition of the equivalent beam during its calculation for the overall longitudinal bending. In such designs of the hull, the reinforced concrete plates of the sheathing were intended only for the formation of a watertight shell of the hull and the absorption of local loads. However, the basic supporting connections of a hull, entirely providing the longitudinal strength, were made of ferroconcrete. An example of the design of a hull representing a combination of keramzit ferroconcrete with reinforced concrete in the incomplete utilization of the latter is indicated in Fig. 24. This design development of a hull of a barge platform with a cargo capacity of 600 tons with a transverse framing system and simplified outlines (flat sides, bottom, and transoms). Within the limits of the cylindrical part of the hull, two longitudinal bulkheads are used to separate the side compartments with a width of 1.7 m and a central compartment 7.6 m wide. The plates of the bottom and the deck within the limits of the side compartments (with a thickness of 80 mm) are made of keramzit ferroconcrete, with concrete reinforcement in a longitudinal direction. These plates, together with the two strong keramzit ferroconcrete longitudinal beams (bottom stringers, separated from the diametrical plane by 1.27 m, comprise the basic support connections of the hull during its overall longitudinal flexure. The total /67

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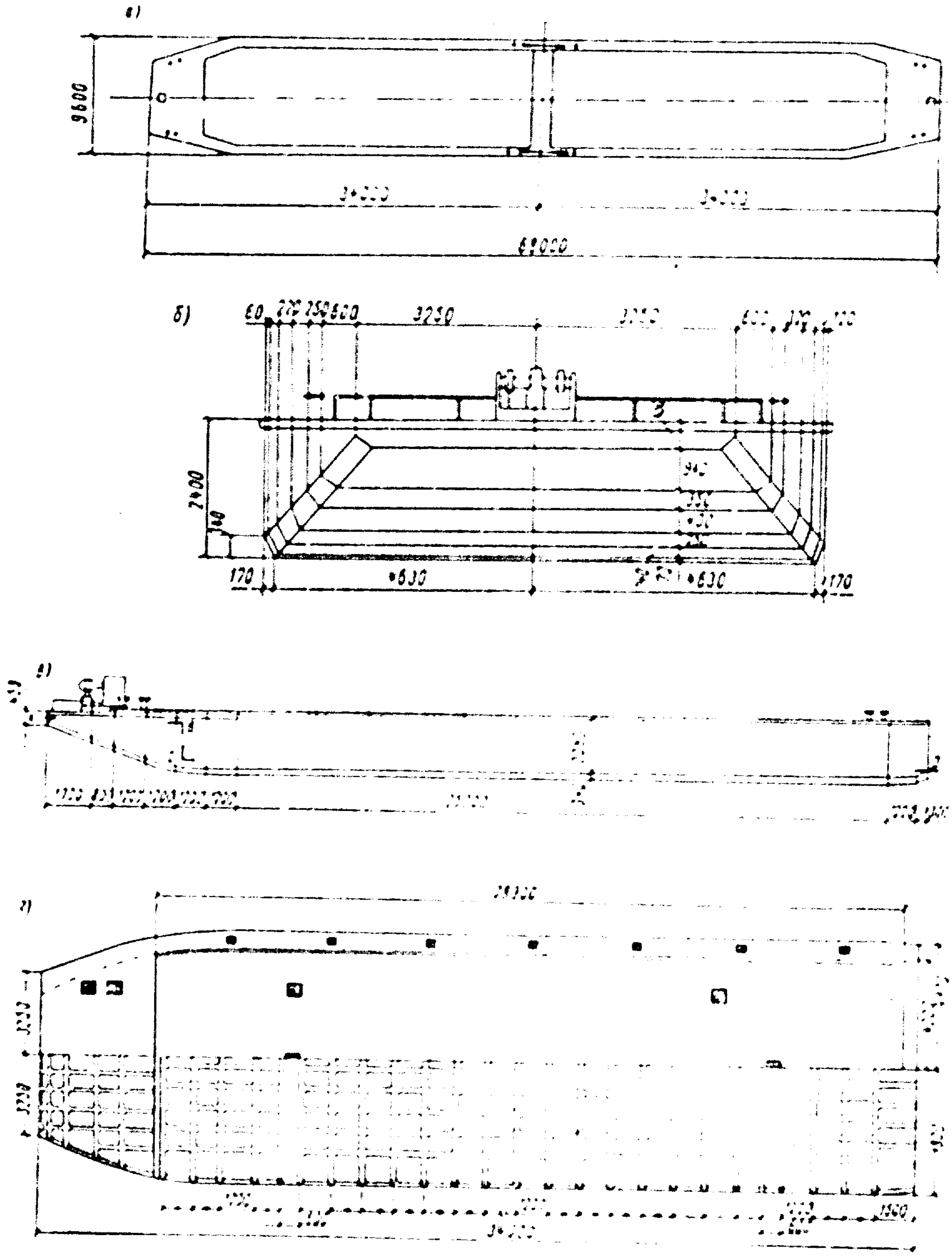

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Fig. 26. Barge with cargo capacity of 1000 tons made of reinforced concrete: a - arrangement of sections in case of coupling; b - view of bow part; c - side view of one of sections; d - plan and section along ABCD of one of the sections.

transverse strength of the barge hull is provided for fairly strong frames, formed by the beams and reinforced bottom frames, and also by the transverse watertight bulkheads, also made of the keramzit ferroconcrete.

The reinforced concrete is used for the plates in the bottom sheathing within the limits of the center compartment, the side sheathing and the longitudinal bulkheads. The depth of the reinforced concrete plates along the side and bottom is 30 mm and along the longitudinal bulkheads, it is 25 mm.

It is natural that such a design of the hull has an excess of strength, and is given here only as an illustration of the unjustified combination of reinforced concrete with ferroconcrete in the composition of marine hulls. Under similar combinations, the reinforced concrete should be taken into account in the calculations of the overall hull strength.

The combination of reinforced concrete with ferroconcrete is also possible in the completely ferroconcrete hulls, where the reinforced concrete can be applied for the production of bulkheads, partitions, platforms and 'tweendecks, which allowed us to reduce the weight of the ferroconcrete hulls. The feasibility of such a combination should be determined with allowance /70 for the production possibilities and the experience of the shipyards in the production of reinforced concrete designs, and also the effect from reducing the hull weight, obtained during the replacement of its individual ferroconcrete designs by the reinforced concrete ones.

Thus, in one of the planning studies of the ferroconcrete hull in a marine ship for purposes of reducing the ship weight to a value assuring its ability to navigate through the internal waterways, the 'tweendeck of the hull was planned to be made of reinforced concrete (Fig. 25). However, in view of the fact that the shipyard builder of this ship did not have experience in producing the reinforced concrete designs and lacked the appropriate

technological equipment, it was considered more economical to make the deck of keramzit ferroconcrete; the production of ship designs from this material had already been mastered by the shipyards, and in its utilization for the 'tweendeck, the required reduction in hull weight had been achieved.

Reinforced Concrete Hulls with Framing in the Form of Flat Reinforced Concrete Membranes. Such a system of framing is also finding application in the hulls made of standard ferroconcrete for ships of small dimensions (for instance, for floating docks 20 m in length).

The hull of the barge platform, of which we spoke previously, has along with the beam framing, the transverse keramzit ferroconcrete membranes in the inter-side space, situated along the frames. As a result of this, the hull within the limits of the cylindrical insert is divided into 38 small side and one central (large) watertight compartments, by which we provide the operating reliability of the barge. For the given barge, in a number of cases, there can be permitted the simultaneous flooding of up to 10 non-adjacent side compartments without risk of losing buoyancy stability or strength of the hull. The framing system in the form of flat membranes in a pure form, i.e. in the absence of any framing beams in the hull makeup, was adopted in the dry-cargo towed barge, built in Czechoslovakia, with a cargo capacity of 1000 tons (Figs. 26 and 27).

Basic Characteristics of Barge

Length, m.	68
Width, m.	9.6
Molded depth, m.	2.4
Sheathing thickness, mm.	30

The barge hull consists of two identically designed hinge-jointed sections with a length of 34 m each.

The lines of the barge hull are simplified and flat. The transom ends are formed by a rise in the flat bottom and the plane sides, with a division into six

transverse sections.

Within the limits of the cargo holds, the hull has a double bottom and double sides, in the space between which are installed the flat longitudinal and transverse membranes (diaphragms). The sheathing of the bottom, sides, second bottom and membranes is made of reinforced concrete 30 mm thick, reinforced by 4 steel gratings of wire 10 mm in diameter, with screen meshes 4 X 4 mm inside size. Along with the finely-meshed screens for reinforcing the plates, and also for supporting the bilge reinforcements and the deck stringer, we use the rod-type reinforcement installed in the center of the plates' depth, between the thin screens. The amount of spacing equal 1200 mm; the distance between the longitudinal ties (membranes) along the bottom is 800 mm.

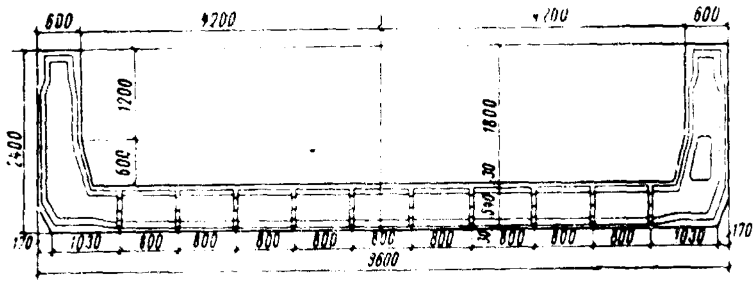

Fig. 27. Design Midship Frame of a Barge Made of Reinforced Concrete.

The method of building the barge hull is sectional-monolithic. The transverse and longitudinal connections (flat membranes) and also the ends of the barge were made separately and delivered to the slipway in the form of finished flat and three-dimensional sections. The preparation of the bottom, the flooring of the second bottom, of the outer and inner sides was achieved directly on the slipway by the monolithic method with the utilization of formless concreting. We should remark that the barge, in distinction from the above-considered reinforced concrete vessels of framing design is completely made of reinforced concrete, including the framing. In this connection, the engineering and design potentialities of reinforced concrete as a shipbuilding material were employed most completely and advantageously in the given design.

Thus, the internal parts of the hull (diaphragms, floors, stringers) represent flat prefabricated (sectional) reinforced concrete designs; the technology of their production is much simpler than that of ribbed plates, consisting of a reinforced concrete strip and of ferroconcrete ribs. From such prefabricated designs, we can assemble the three-dimensional sections and deliver them to the slipway.

The division of the barge hull into two hinge-jointed sections creates the conditions under which we reduce appreciably the expanding (stretching) forces from the overall longitudinal flexure, which is quite important for /72 reinforced concrete having a relatively low permissible stress for expansion, which essentially limits the dimensions and cargo capacity of the reinforced concrete vessels. It is evident that such design measures will also be useful in the development of reinforced concrete barges of large cargo capacity, at the same time facilitating the expansion of the range of efficient utilization of reinforced concrete in shipbuilding.

The production of the cylindrical part of the hull directly on the slipway by the monolithic method with the use of a formless concreting should also be regarded as a technically and economically justified solution, in spite of the general tendency of ferroconcrete shipbuilding toward the completely prefabricated construction method. It appears that the formless method of building the reinforced hulls, since it significantly raises the technical-economic indexes of the monolithic method of construction, should find that broad application in the building of ships of reinforced concrete, especially prior to the development of highly-productive equipment for the preparation of prefabricated marine reinforced concrete designs, and the finding of more simple and technologically efficient intersectional connections.

Section 12. Design of Marine Reinforced Concrete Plates.

A basic element of the reinforced concrete hulls and superstructures of ships is the design consisting of plates reinforced by the framing. In this connection, the plates are the reinforced concrete itself in the composition of the hull, whereas the framing beams are made chiefly of ferroconcrete. The total weight of the plates in the composition of the hull comprises more than half the hull weight.

The reinforced concrete plates in use as hull sheathing can be flat or curvilinear, while depending on the hull design, they can be ribless or with a framing. In their purpose in the makeup of the hull, the plates are supported by ribs, running only in one direction or mutually intersecting (Fig. 28).

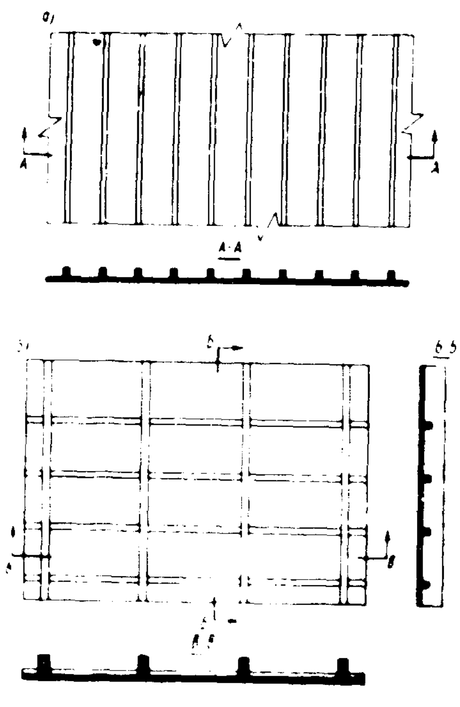

Fig. 28. Reinforced concrete plates with ferroconcrete ribs: a - arrangement of ribs in one direction; b - arrangement of ribs in two directions.

The plates in use for bulkheads, and partitions in the hull and superstructure and also for the wall panels and covers of the superstructure can be spatial reinforced concrete elements of the shell or fold (convolution) type.

The thickness of the reinforced concrete plates of the hull sheathing, as the ferroconcrete ones is determined from the condition of adequate strength with the participation of the ship hull in the overall bending and in the absorption of local loads (the minimal thickness of reinforced concrete plates should comprise not less than 10 mm, while the maximal should be 50 mm). Depending on the plates' thickness, their reinforcement can be made as only thin finely-meshed screens, or as thin screens in combination with the usual reinforcement in the form of rods or of rod-type screens, located in the central part of the plate's section height. /73

It is recommended that the plates with a thickness of 10 - 15 mm be reinforced only with thin woven or welded screens.

For the plates thicker than 20 mm, basically we apply the combined reinforcement (the thin screens in combination with the rod reinforcement in the form of individual rods or welded screens).

The reinforcement of plates and their thickness are determined by strength calculations and are conducted in accordance with the recommendations discussed in Chapter I.

The number of layers of woven screens in the reinforced concrete plates, from the condition of providing the guaranteed quality of the placing and packing of concrete in the design, is assumed to be not more than:

For plates with a thickness

10 mm .... 4 layers
15 mm .... 6 layers
20-30 mm .... 8 layers

In this connection, for the plates having a combined reinforcement, from the side of the expanded zone, we use not less than 2-3 thin screens.

The diameter of the rod-type reinforcement in the combined strengthening of the plates is established in dependence on the plates thickness, from the

condition of installing in the plate the required number of woven (fabric) screens, i.e. the obtainment of the specific reinforcing surface required for the shipbuilding reinforced concrete, and also for providing a protective concrete layer 2-3 mm in thickness. Under these conditions, as a rule the diameter of the reinforcing rod is not more than 5-6 mm.

The number of rods per running meter of intermediate screen is established by calculation and should correspond to the requirements of the "Rules for Construction of Ferroconcrete Ships" of the River Register of the RSFSR, according to which the number of rods in the working direction of the plate (the rods /74
directed parallely to the smaller side of the support edge of the plate) should be not less than 5 and not more than 20 per running meter of the screen. The sectional area of the distributing reinforcement in the plates (rods, directed parallely to the larger side of the plate's support edge) should be not less than 15% of the sectional area of the working reinforcement. In this connection, the distance between the rods, from the condition of standardizing the reinforcing rods, should be a multiple of 50 mm.

The depth of the protective layer of concrete (independently of the purpose of the reinforced concrete plates) in the composition of the hull and the superstructure of the ship falls in the limits of 2-3 mm. The contact of the thin screens with each other during the fabrication of the reinforcing frames of the plates can be conducted without lap welding with the overlapping of the ends by not less than 100 mm. In the application of resistance-spot welding, the overlapping of the ends is not less than 30 mm. The joints of one layer of screen should be displaced relative to the joints of the other layers so that in any section of an element, there would not be more than one joint. The connection of the rods of the intermediate screens is conducted as in the designs made of ordinary ferroconcrete.

In distinction from the ferroconcrete plates, as a rule the reinforced concrete plates have a reinforcment section which is symmetrical relative to the CG (center of gravity). However, in certain instances for increasing the strength of a plate operating under flexure with an expansion, the expanded zone of the plate in the working direction is reinforced additionally with rods having a diameter of not more than 5 mm.

Since the plates of the reinforced concrete hull are calculated as beam-strips, fitted into the support, where the bending moments are considerably higher than in a span, the additional reinforcement by rods can be utilized for complying with the strength conditions on the support. Owing to this, the plate thickness is reduced appreciably.

Section 13. **Framing Beams**

The framing beams of the hulls and superstructures of the ships made of reinforced concrete can be of ferroconcrete (of the ordinary or prestressed ferroconcrete) and metal.

The design developments and the experimental studies on establishing the possibility and feasibility of applying the framing beams of reinforced concrete (with combined reinforcement, and reinforced only by woven screens) have shown that the introduction of woven screens into the plate ribs complicates greatly the procedure in producing the ribbed design and does not guarantee their quality. In connection with this, the framing beams made of reinforced concrete have in effect not found application. /75

The simplest from an engineering standpoint are the framing beams made of ordinary ferroconcrete, which have also found wide acceptance in the construction of reinforced concrete hulls. In the fabrication of the ribbed reinforced concrete plates, the ferroconcrete framing beams are concreted with cement-sandy concrete of the same composition. Under the separate technology of fabricating the plates

and r..bs, it is also possible to have the concreting of the ribs by keramzit concrete in order to reduce the weight of the designs.

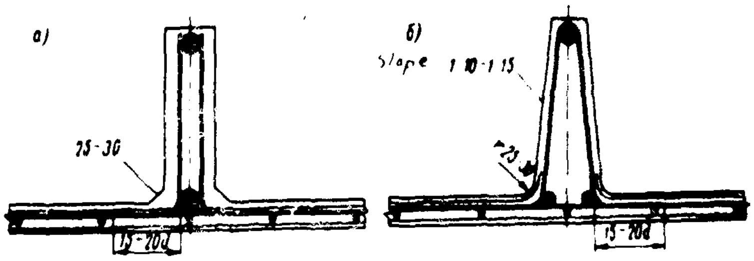

Fig. 29. Configuration of cross section of ferroconcrete beams strengthened by reinforced concrete plates: a- in the fabrication of the designs with beams upward; b - in the fabrication of designs with the beams downward.

The configuration of the cross section of ferroconcrete framing beams depends on the technique of fabricating the ribbed designs of the plates. In the production of the plates with ribs upward, the cross section of the ribs is a rectangle (Fig. 29, a); in the production of plates with ribs downward, in the nondetachable form-matrices, the lateral surfaces of the framing beams have a slope ranging from 1:10 to 1:15 depending on the beam height (Fig. 29, b).

The dimensions of the framing beams and their reinforcement are established from the conditions of providing the strength, minimal weight and technological effectiveness of the designs. The approximate sizes of the framing beams are adopted in the following multiple of the dependence on the thickness of reinforced concrete plate: with 2 - 2.5; height 5 - 8.

As a rule, the ferroconcrete framing beams have chamfers in the places of attachment to the plate, with a cathetus value of 25-30 mm.

It is recommended that the reinforcement of the beams be accomplished in the form of flat or three-dimensional welded frames. The diameter of the working reinforcement of the beams is not less than 6 mm, while the inside distance between the reinforcing rods in one direction be not less than 5 mm.

The clamps are made from reinforcing rod with a diameter ranging from 4 to 6 mm according to the diameters of the intermediate rod reinforcement

in the reinforced concrete plate. The distance between the clamps equals the distance between the rods of the intermediate plate reinforcement, installed perpendicularly to the beam's axis. /76

The thickness of the protective concrete layer of the framing beams measured from the outer surface of the clamps to the outer beam surface is not less than 5 mm.

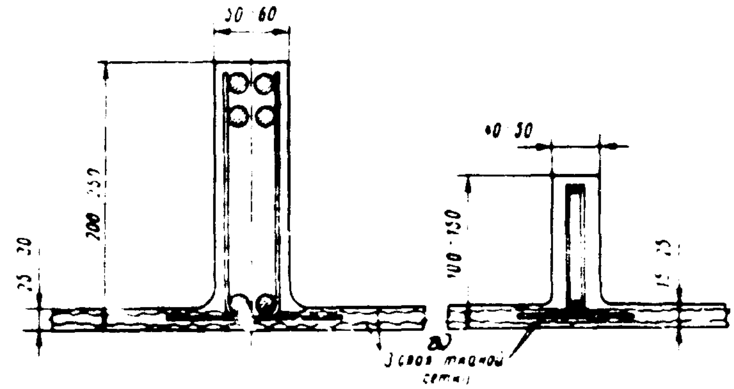

Fig. 30. Systems of Reinforcing the Ferroconcrete Framing Beams of Reinforced Concrete Designs Key: a) 3 layers of woven screen.

The cross sectional area of the working beam rods is established by the strength calculations, and the number of rods is also found based on the conditions of their installation in the beam, i.e. by the dimensions of its cross section. Thus, the beams of small sizes (50 X 150 mm) as a rule have in the lower and upper part one working rod each of about the same diameters (Fig.30). For the beams of larger dimensions (60 X 250 mm), the number of rods in one row along the horizontal is doubled, and when necessary, in the upper part of the beam, the rods are installed in two rows (see Fig. 30). With such a number of rods, it is quite important to maintain between them along the vertical and horizontal an interval required for providing the combined operation of the reinforcement with the concrete.

The observance of the indicated spacings in the units of the intersection of beams of different direction is practically impossible, therefore in the intersection points, the requirement concerning the preservation of the standard

spaces between the rods is extended only to the rods running parallely; as a rule, the intersecting rods come into contact (Fig. 31).

The connection of the framing beams with the reinforced concrete plate should be realized by way of diverting the bent parts of all clamps of the beam between the woven screens (in the reinforcement of plates, only with woven screens) or beyond the intermediate rod-type screens (in case of the combined reinforcement of plates). The length of the clamp folds anchored in the plate is not less than 15-20 diameters of the clamp. /77

Such a design of connecting the reinforced concrete plates with the ferroconcrete framing beams was verified during the testing of beams with a connected strip for bending. All of the beams broke along the oblique cracks without a shifting of the plate relative to the beams, which confirms the reliability of the connecting system adopted.

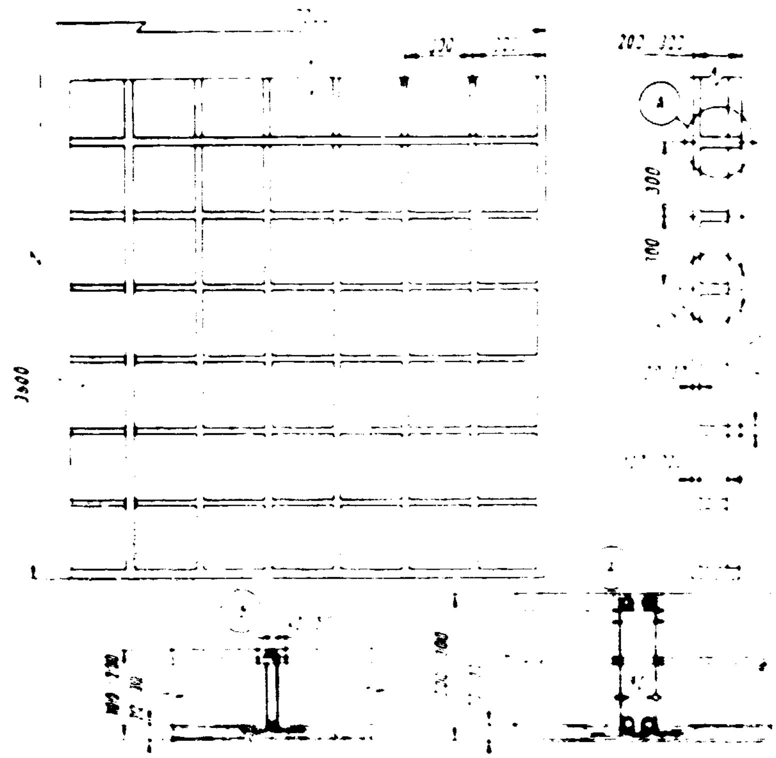

Fig. 31. Intersecting Ferroconcrete Framing Beams of Reinforced Concrete Designs.

The connection of the fittings of the ferroconcrete beams along the length, and the construction of the intersections of beams in varying direction are conducted in conformity with the instructions of the "Rules for the Construction of Ferroconcrete Ships" of the River Registry of the RSFSR by analogy with the designs made of standard ferroconcrete.

For purposes of reducing the difficulty in producing the hulls and superstructures of ships from reinforced concrete, it is necessary to strive toward the maximally possible standardization of the dimensions and reinforcement of the frame beams. /78

Section 14. Intersectional Joints

The application of the prefabricated method of building ships of reinforced concrete involves the necessity of connecting the sections or units during the assembly of the ship on the slipways. These intersectional joints have their unique features.

Connections in one plane. The major part of the intersectional joints occur in the hull plating.

The main requirements which the connections of the sheathing plates of the reinforced concrete hulls must meet include the provision of strength, watertightness and technological effectiveness. The primary type of connecting the sheathing plate in one plane is the connection in the outlets with a bypass of the ends (extruding from each section) of the fiber-type and rod-type intermediate grids.

Depending on the type of load on the design, the connections of the plates on the outlets can be made without welding or with welding.

The weldless joining of the plates is used only for the designs operating on contraction of flexure. In this case, the value of the by-pass of the fiber grids is not less than 50 mm, while the size of the by-pass of the intermediate rods is not less than 20 diameters of the rods (Fig. 32).

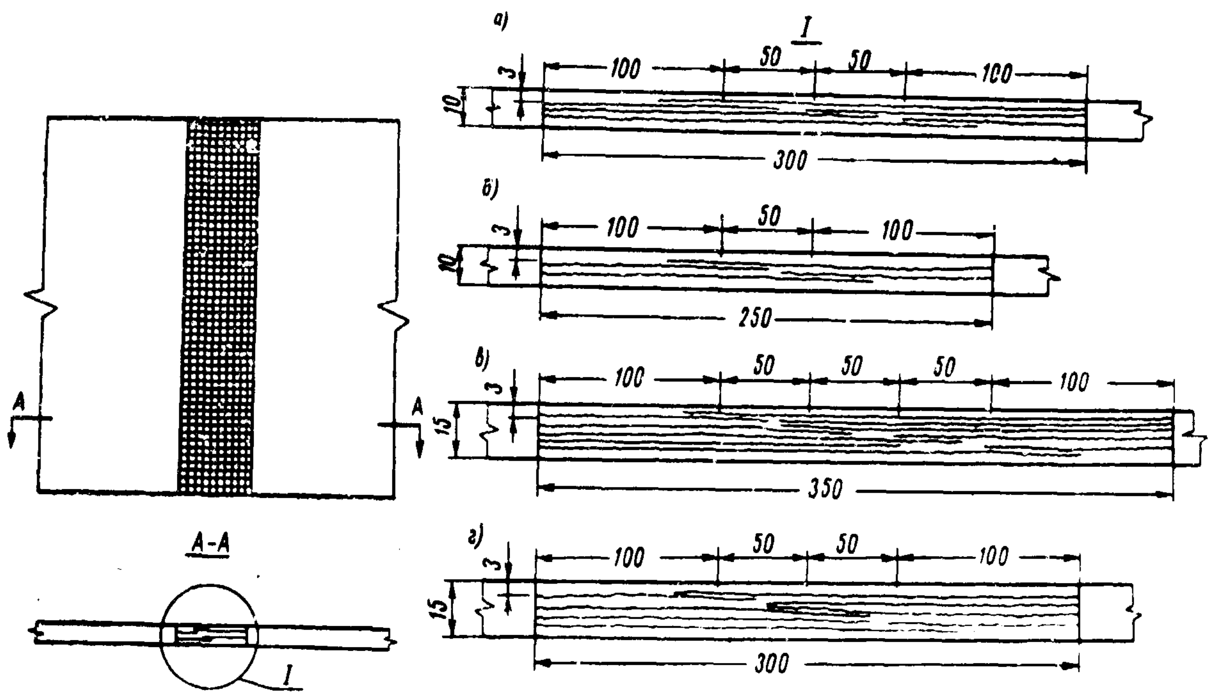

Fig. 32. Examples of the Connection of Reinforced Concrete Plates with a Thickness of 10-15 mm for the By-pass of the Webbed Screens During Reinforcement: a-with three; b- with two; c-with six; and d-with four webbed screens.

The connections of the plates on the outles for the designs operating on axial and eccentric extension, are made with the welding of the by-passes of the webbed screens and of the intermediate rods. The amount of the by-pass of the webbed screens in case of their joining by resistance spot welding is not less than 20-30 mm, while the by-pass of the intermediate rods (and the length of the welded joint of the rods) is not less than 10 diameters of the rod (Fig. 33).

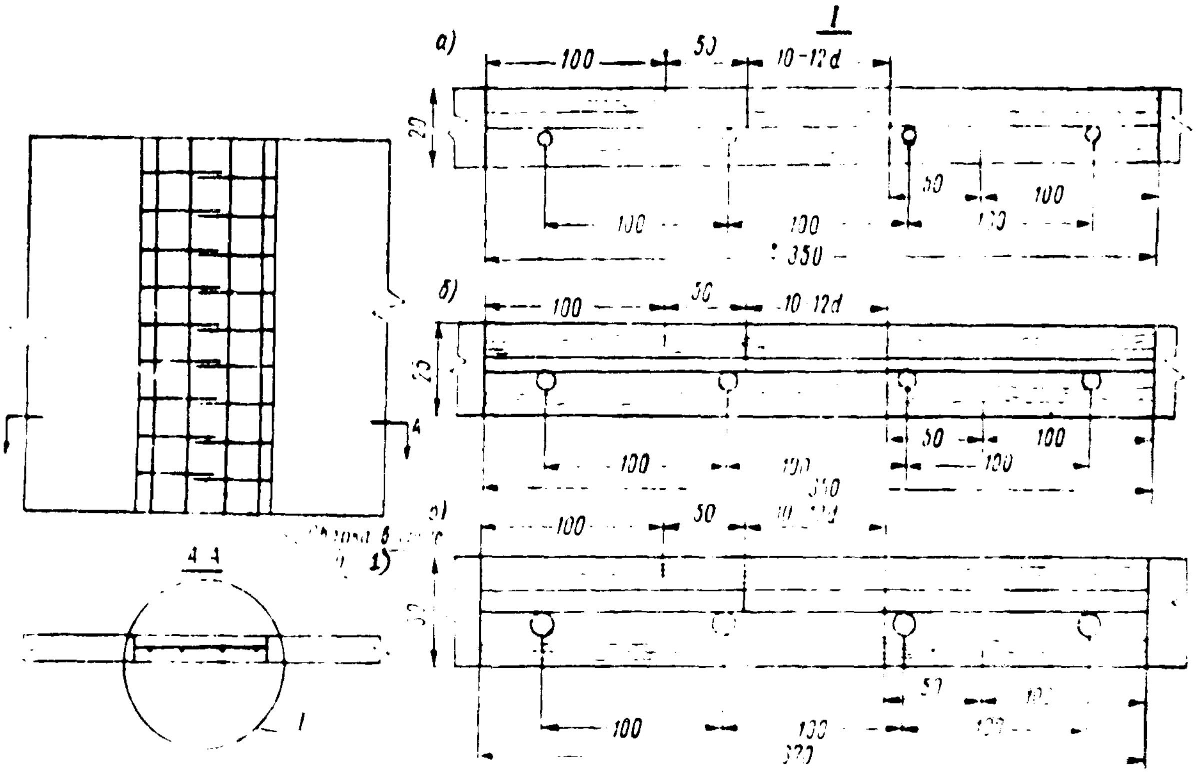

Fig. 33. Examples of Joining the Reinforced Conc ete Plates with a Thickness of 20 - 30 mm in the Welding of Outlets during Reinforcement: a - with four; b- with six; c- with eight webbed screens and intermediate rod-type screen. Key: 1)welding in at atmosphere of CO_2.

In both cases, the joining of the webbed screens (both with welding and without welding) is done in order that there would by-pass flush all the screens, and in each section there would not be more than two joints of the screens.

The types of connections shown in Figs. 32 and 33 are equivalent with the monolithic type in strength and crack resistance, since the structure of the reinforcement does not disrupt in the joints, which has been confirmed by specially conducted tests. However, the connections of such a type are laborious in preparation, in view of the large volume of rigging and welding activity. and also the complexity of making the joint monolithic, especially in

a vertical position.

In spite of the indicated technological disadvantages of this connection, it should be applied for the outer sheathing of the hull, since the crack resistance, and hence the watertightness and the corrosion strength of the joint are equivalent to that of the monolithic type.

The connection which is most technologically simple is the joining of plates including the welding of the by-pass intermediate rods, but without by-passing in the zone of connecting the fine networks (screens)(Fig. 34).

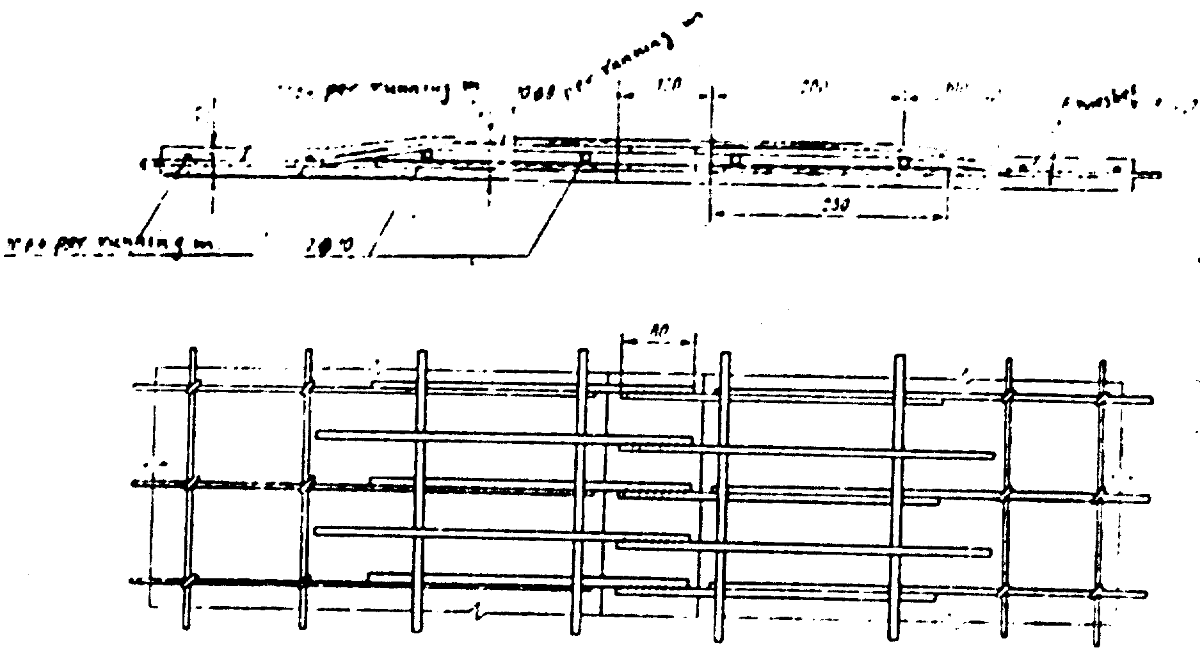

Fig. 34. Connection of Reinforced Concrete Plates with Welding of the Outlets. Key: a)per running meter; b) six meshes N 8-0.7

For providing equal strength of the joint with that of the /82
monolithic type, in the plates along their edge during the preparation, we install additional rod-type fittings, the diameter and number of which are established by calculations. The additional rod-type fittings are anchored in the prefabricated designs for a

length of not less than 20 diameters. In accordance with the amount of the additional rod-type fittings being introduced, the thickness of the joining edges of the prefabricated designs is increased somewhat. For the purpose of reducing the settling stresses of the concrete in the joints, there is achieved the butt-type by-passing of at least one webbed netting.

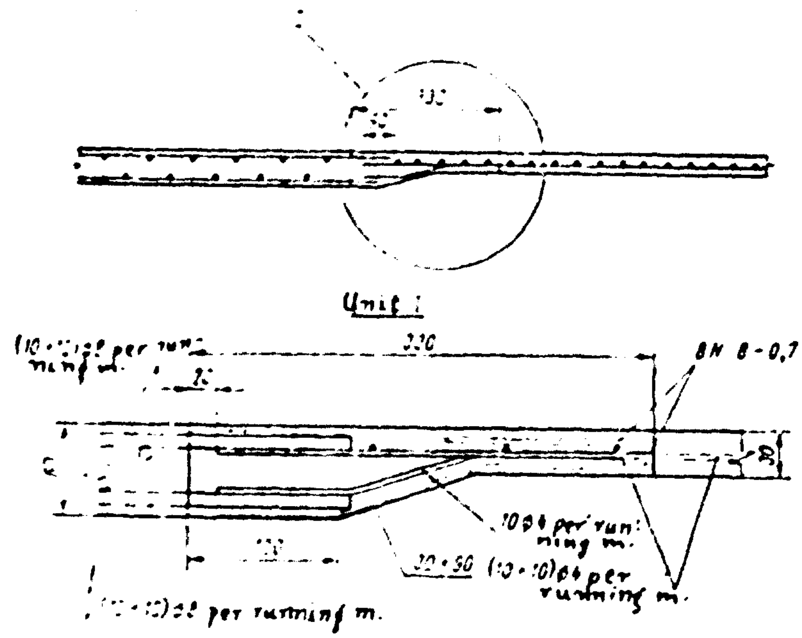

Fig. 35. Connection of the Reinforced Concrete and Ferroconcrete Plates. Key: a) unit 1, b) per running meter.

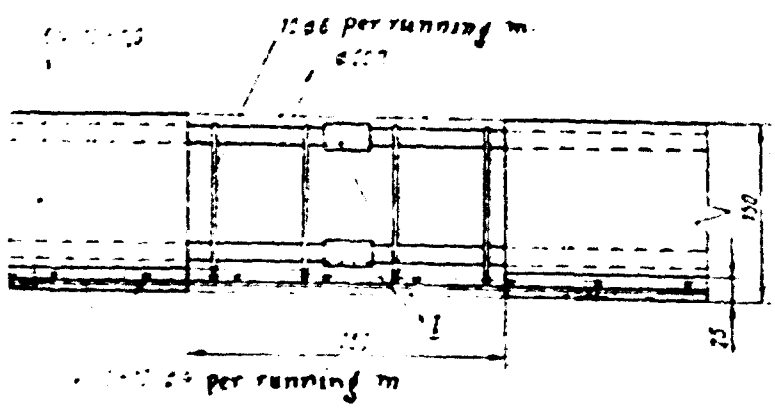

Fig. 36. Connecting the Framing Beams. Key: a) per running meter.

The given type of joining the reinforced concrete plates does not guarantee a crack resistance of the joint equivalent with that of the monolithic type, since in the joint, there is concentrated reinforcement, while in the monolith, the reinforcement is scattered (although in the extreme fibers). A major disadvantage of this joint is the necessity of increasing the plate thickness along the connected edges and along the joint itself.

In connection with this, the given type of joint can be recommended only for the designs of the superstructures, deckhouses, partitions, bulkheads and platforms.

In the case of applying in the composition of the hull or the superstructure, along with the reinforced concrete plates, of plates made of standard ferroconcrete (e.g. in the use of reinforced concrete in ships made of ferroconcrete, as platforms, partitions, light decks or for the superstructures), the interconnection of the plates is conducted according to the system indicated in Fig. 35.

The design of the units for connecting the framing beams of the reinforced concrete plates basically does not differ from the design of connecting the framing beams of the ferroconcrete plates. It is necessary only to keep in mind those specific features in the reinforcement of the framing beams which are discussed in Section 13.

We have indicated in Fig. 36 the joining of the ferroconcrete framing beams of the reinforced concrete plates in one plane.

Corner tee and four-way joints. Among the corner joints, we include the connections of the bulkheads with the bottom, with the deck, the sides and of the sides and transoms with the deck and the bottom.

All the corner-type joints can be subdivided into two main types:

1) joints in the fitting outlets (of the webbed networks, of the intermediate rods and of the additionally introduced rods-anchors) with welding or without welding of the outlets; and

2) the joints in the inserted parts with a welding of the protruding (along the connecting edges) inserted parts.

The requirements for the corner joints in the part of the length of the by-pass of the screens and of the rods, the lengths of the welded seam and the arrangement of the reinforced metal of the screens is the same as those imposed on the flat connections.

For the designs of the outer contour of the hull, we apply chiefly the joints of the first type: of the side framing with the bottom (Fig. 37); of the side plates with the bottom plates and the deck (Figs. 38 and 39); of the bulkhead plates with the bottom plates (Fig. 40); of the deck framing with the bulkhead framing (Fig. 41). The connections on the fitting outlets are also used in the units of the intersection of three sections (Fig. 42).

In this connection, as in the flat joints, we use two /89 different modifications of the corner joints in the fitting outlets: with by-pass and without by-pass of the webbed screens in the contact zone.

For the designs of the external shape of the hull, one should prefer the joints with the by-pass of the webbed screens in the contact zone, since this increases the crack resistance of the joint.

The joints based on the welding of inserted parts are used mainly

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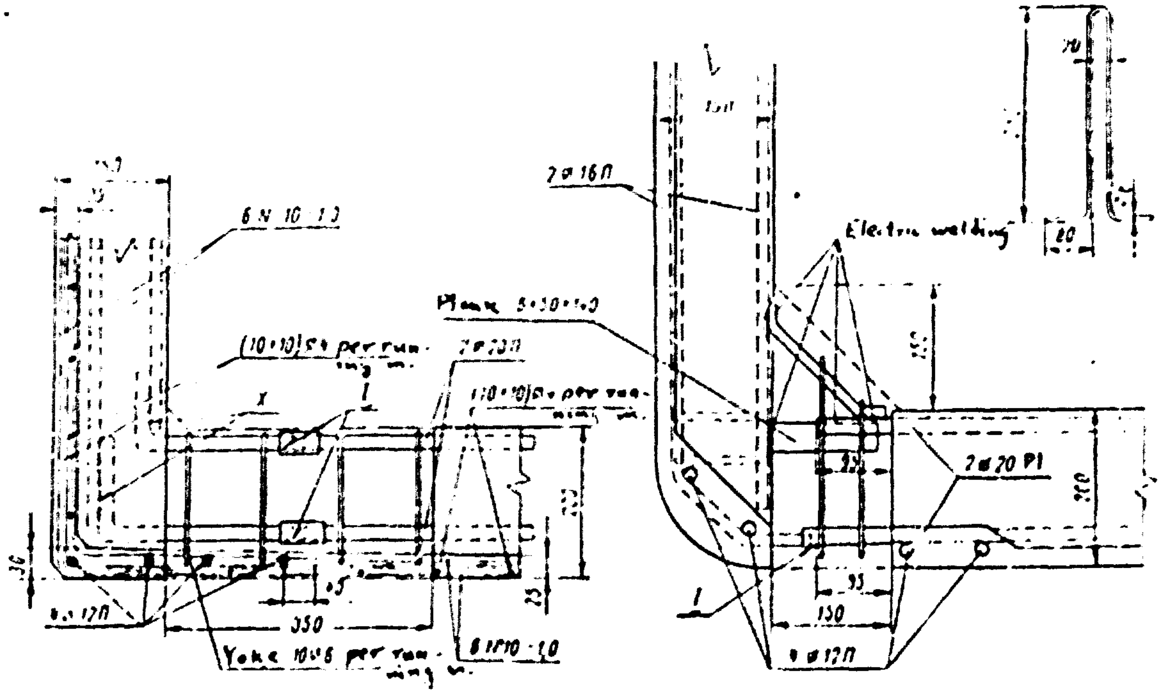

Fig. 37. Examples of Joining the Side Frames with the Bottom Frames.

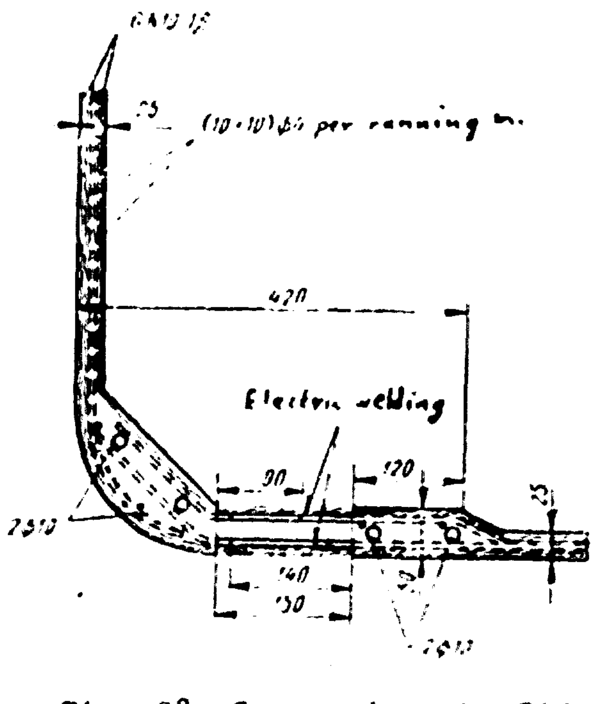

Fig. 38. Connecting the Side Plates with the Bottom Plate.

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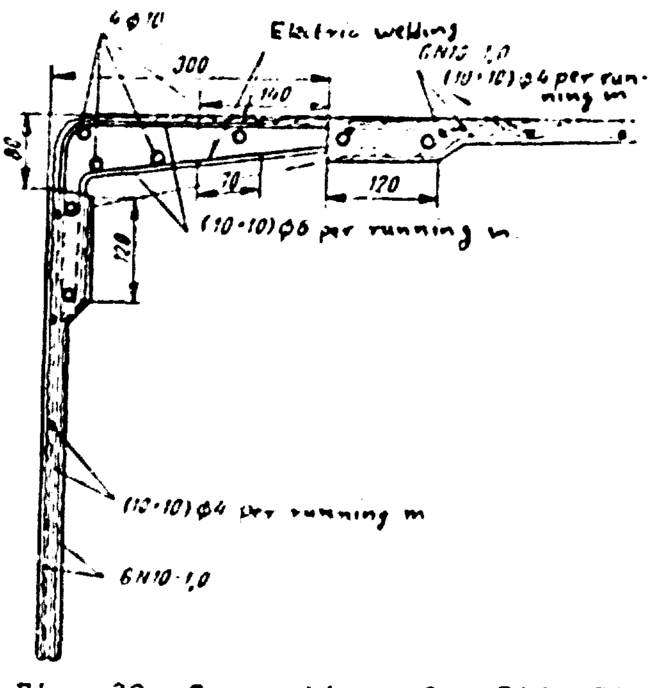

Fig. 39. Connection of a Side Plate with the Deck Plate.

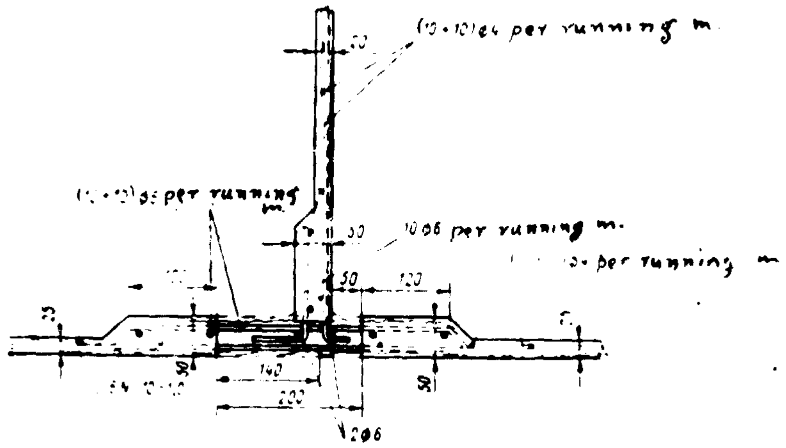

Fig. 40. Connecting the Bulkhead Plates with the Bottom Plates. /86

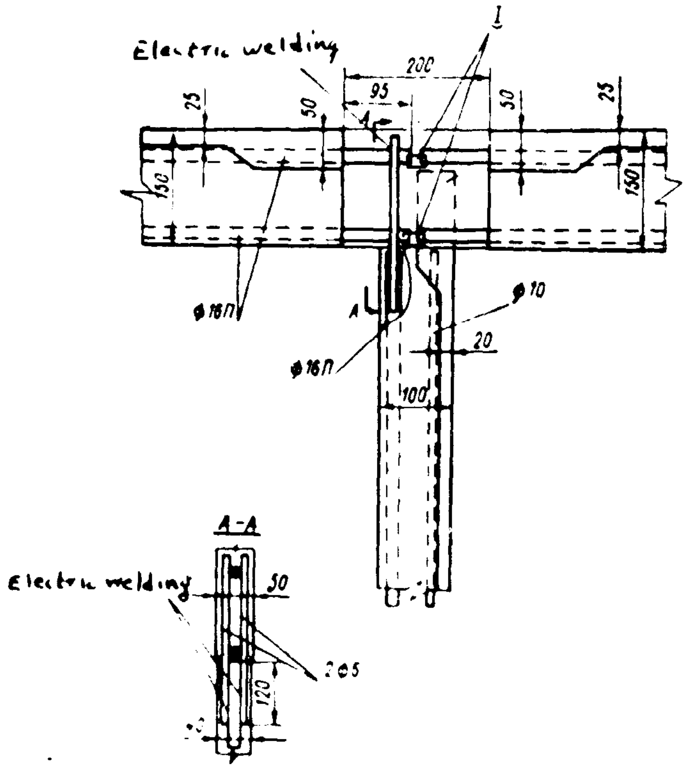

Fig. 41. Connecting the Ribs of the Longitudinal Bulkhead with the Beams.

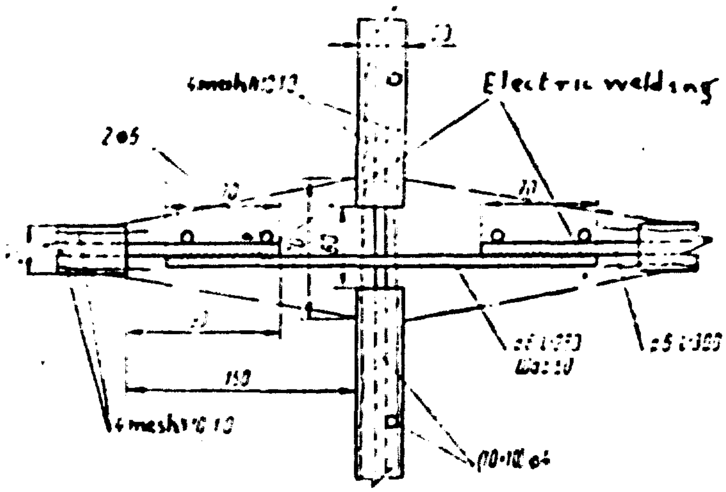

Fig. 42. Connecting the Longitudinal and Transverse Bulkheads on the Fitting Outlets.

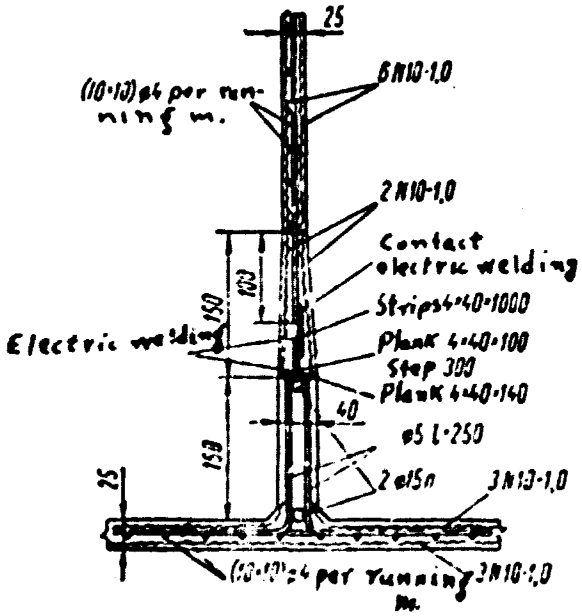

Fig. 43. Connecting the Bulkhead Plate with the Bottom Framing.

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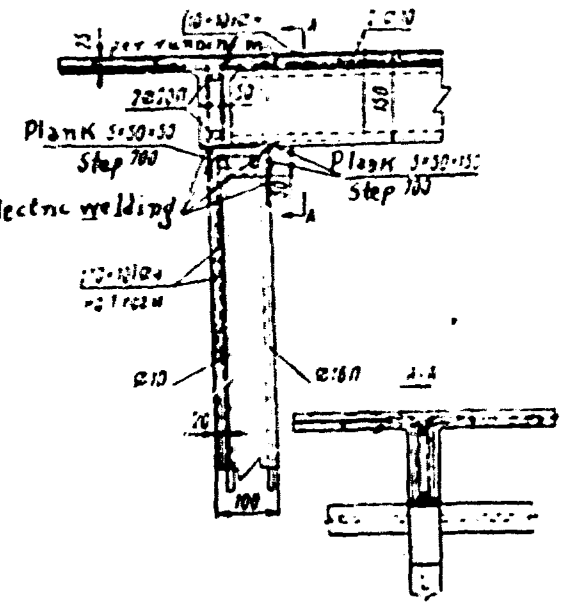

Fig. 44. Connecting the Bulkhead Sections with the Deck Framing.

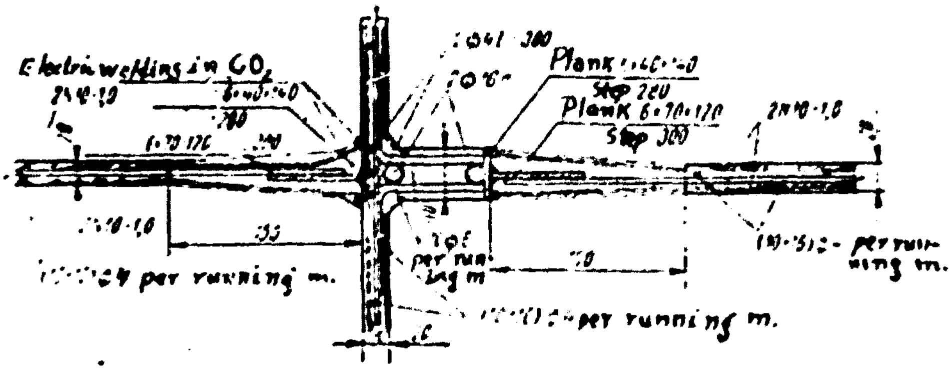

Fig. 45. Connecting the Longitudinal and the Transverse Bulkheads on the Welding of the Inserted Parts.

/89

for the designs of the inside contour of the hull: the joints of the bulkhead plates with the rib of the deck section (Fig.44); of the longitudinal and transverse bulkheads and the connection of the bulkheads with the sides and with the transoms in the case of the contact of one of the bulkheads to the rib of the other (Fig.45).

In the corner joints based on the welding of the inserted parts, just as in the flat connections, for purposes of providing the adhesion of the concrete with the reinforcing metal, in the joint zone, one-two webbed mesh-wires are by-passed.

The connection of the walls of the superstructure with the deck is also made on the basis of welding the inserted parts (Fig. 46).

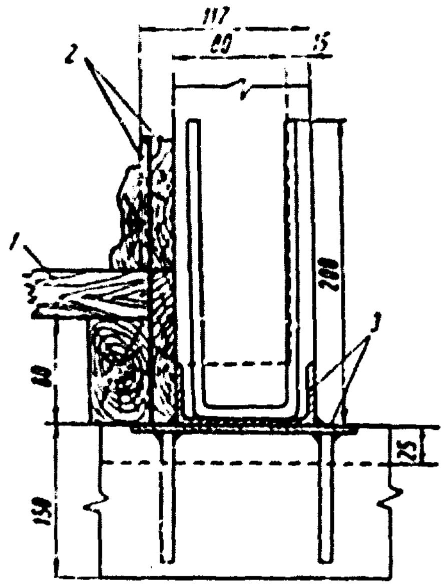

Fig. 46. Connecting the Walls of the Superstructure with the Deck. 1-decking of the superstructure floor; 2-sheathing of the area (of board, tolite-skin, plywood); 3-the inserted parts with the anchors.

The interconnection of the superstructure walls and of the internal elements of the superstructure can be made on the basis of welding the inserted parts similarly to the way that the joints of the designs of the internal contour of the hull are made.

The corner joints of the reinforced concrete ribbed designs with the ferroconcrete ones are made on the basis of welding the inserted parts with the by-pass of the mesh-wires in the

contact zone of the reinforced concrete plate with the ferroconcrete rib (Fig.47).

In the case of the application of steel framing beams or beams made of prestressed ferroconcrete, the design of the plates' joints will remain the same as in case of the ferroconcrete beams. The interconnection of the framing beams is accomplished in these cases as in the designs made of prestressed ferroconcrete or of steel. /90

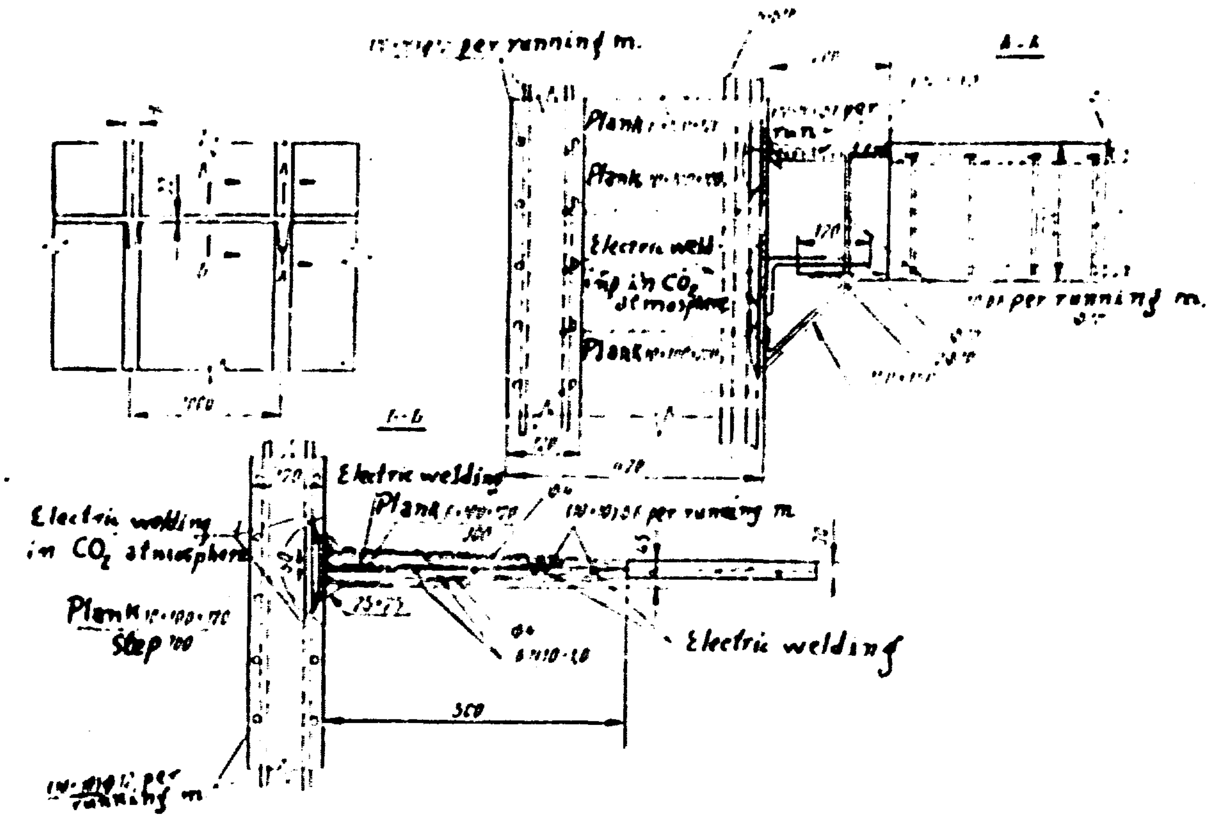

Fig. 47. Angular Joint of Ferroconcrete and Reinforced Concrete Ribbed Designs.

Section 15. Reinforcement to the Reinforced Concrete Hull of Internal Saturation of Ship.

The attachment to the reinforced concrete designs of the hull and the superstructures of internal saturation (mechanisms, devices,

systems, rigging and equipment of the quarters) is achieved either by means of inserted parts or with the aid of through bolts passed through the design. The attachment with the aid of through bolts is accomplished only to the designs not experiencing a constant water pressure. The conduct of the attachment with the aid of through bolts to the reinforced concrete is much simpler than to ferro-concrete, since the hulls in the reinforced concrete plates can be drilled on the spot.

The basic type is the attachment by means of inserted parts. The inserted parts can be installed in the reinforced concrete plate and in the ferroconcrete framing beams. The attachment of the inserted parts to the reinforced concrete plates can be accomplished by welding to the rods of the intermediate mesh-wire, to the rod clamps, additionally inserted into the mesh-wire (Fig.48), and also to the special anchoring rods (Fig.49).

The type of the attachment of the inserted parts to the plates indicated in Fig.48, is preferable from the viewpoint of weight of the design, however this joint operates much less efficiently to cleavage than indicated in Fig.49. At the same time, the type of attachment indicated in Fig.49, providing the possibility of absorbing considerably higher separating forces, requires a local thickening of the plate, which is inefficient from the viewpoint of the weight of the design and the technology of its manufacture. Therefore, if possible we should avoid the attachments to the reinforced concrete plates of the saturation parts of the hull, causing considerable separation forces, and conduct the attachment of such parts to the framing beams (Fig.50). For the parts of the filling of the hull, not causing any appreciable separation forces,

the inserted parts are fastened according to Fig.48.

The examples of attaching the inserted boxes to the reinforced concrete plates (with the use of the design solutions presented in Fig. 49) are shown in Figs. 51-53.

All of the inserted parts intended for the attachment of the internal filling of the ship to them, by degree of load state can be subdividided into three main groups:

1) the inserted parts for the attachment of the main and auxiliary engines, the deck mechanisms and devices;

2) the inserted parts for the attachment of the systems and pipelines; and

3) the inserted parts for the attachment of the lightly-loaded elements of saturation (electric lines, cable lines, the heating devices, the lighting fixtures, etc). /92

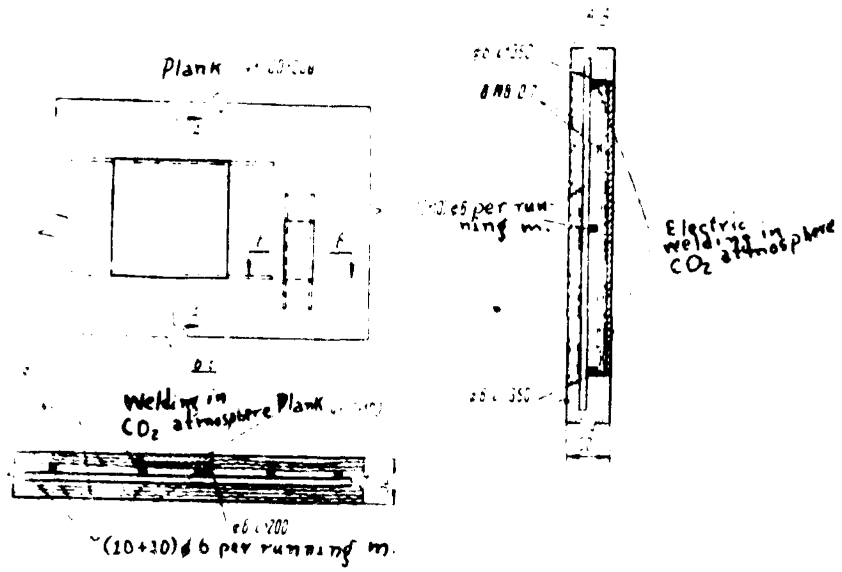

Fig. 48. Attachment of the Inserted Cleats (planks) to the Reinforced Concrete Plates by Welding to the Reinforcements of the Intermediate Mesh.

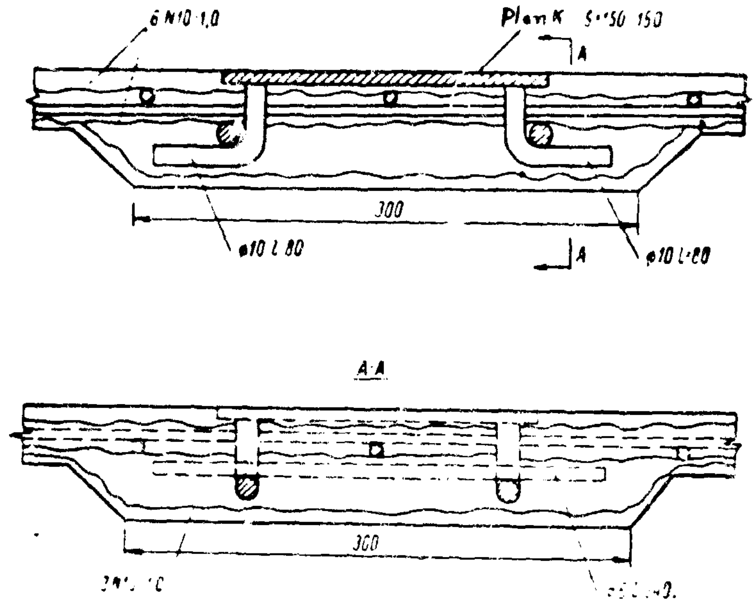

Fig. 49. Attachment to Inserted Strips to the Reinforced Concrete Plates by Welding to the Anchoring Rods.

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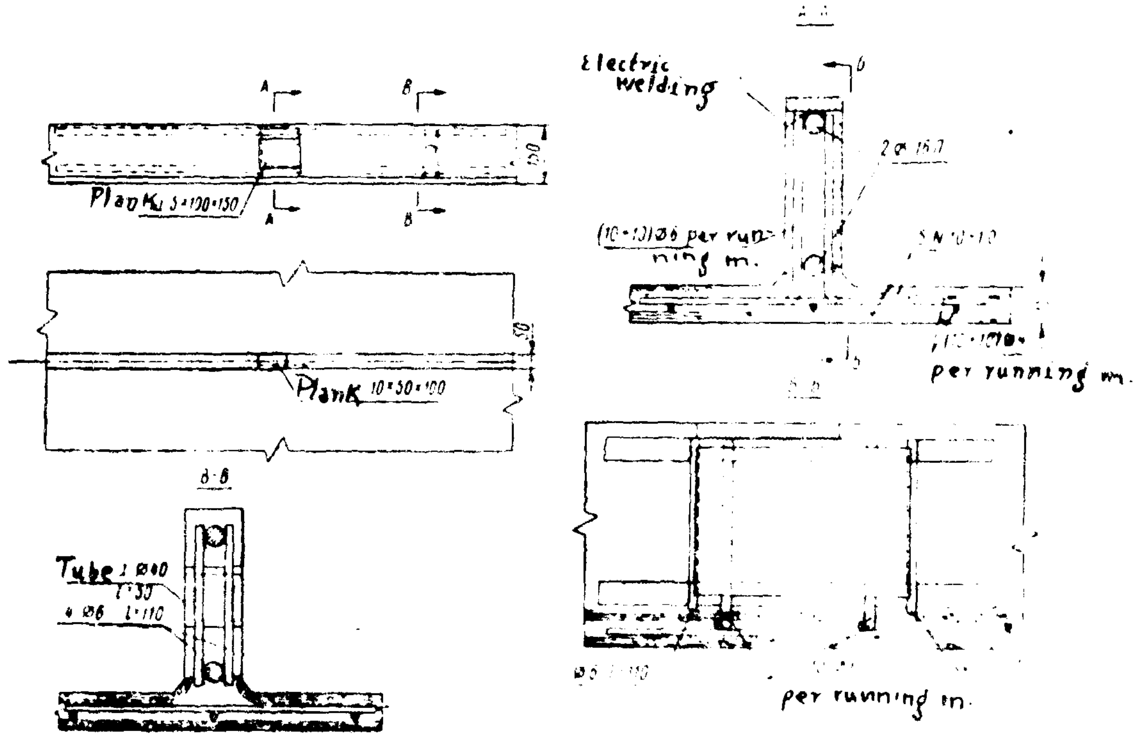

Fig.50. Fastening the Inserted Parts to the Framing Beams.

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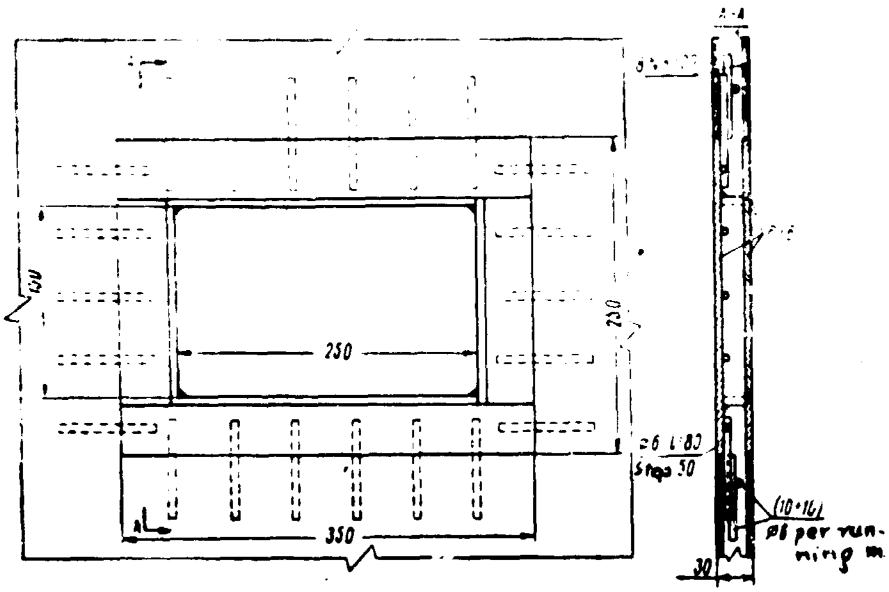

Fig.51. Fastening of the Rectangular Inserted Box.

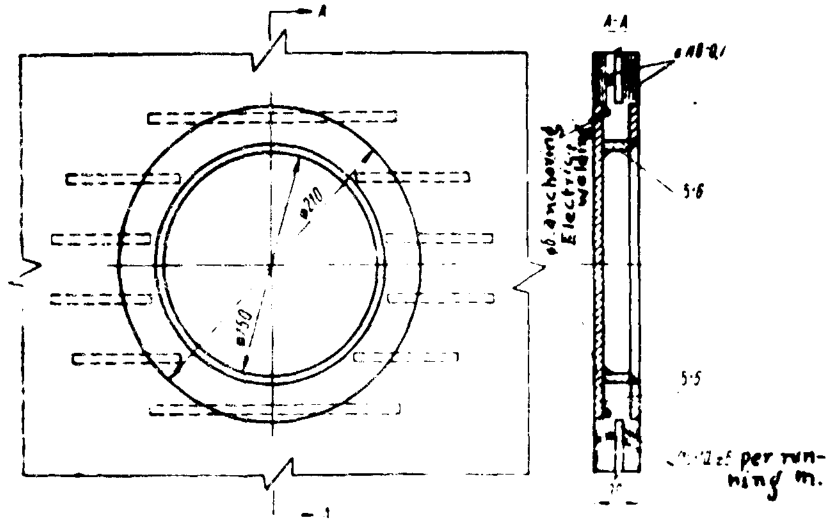

Fig. 52. Attachment of Round Inserted Box. /95

The main and auxiliary engines are installed on foundations, often formed with inserted stiffening ribs. For this purpose, we use the framing beams and the specially installed ribs. The

attachment of the engines to the base plate is achieved by means of inserted strips, welded to the anchoring rods (specially mounted in the stiffening ribs) and to the fittings of the stiffening ribs (Fig.54). The strength and the stiffness of the engines' mounting in the design of attachment shown in Fig.54, is determined by the strength of the anchoring of the inserted strip-cleats and the rib's stiffness. The dimensions and reinforcement of the stiffeners and the dimensions of the anchors are established from the condition of the absorption by them of the dynamic load from the operating mechanisms.

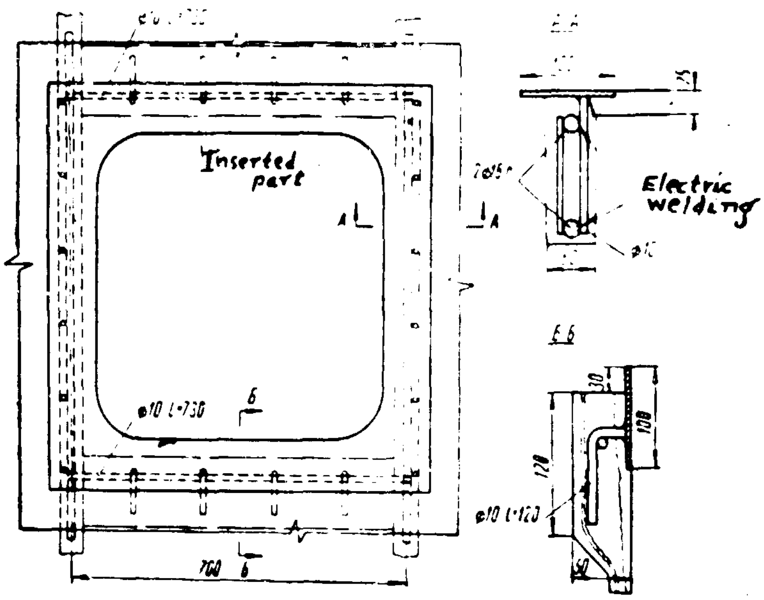

Fig.53. Attachment of the Inserted Part under the Hatch Coaming.

To reduce the virbrations transmitted from the mechanisms to the hull, between the base frame of the mechanism and the ferro-concrete foundation,shock-absorbing linings are installed.

The deck mechanisms and devices can be attached to the sheathing plates (Fig.55) and to the framing beams. Allowing for the slight thicknesses of the reinforced concrete plates, and hence the low stiffness of the plates, the vessel devices and the deck mechanisms, transferring considerable loads to the hull, should also be attached simultaneously both to the plates and to the framing beams. /96

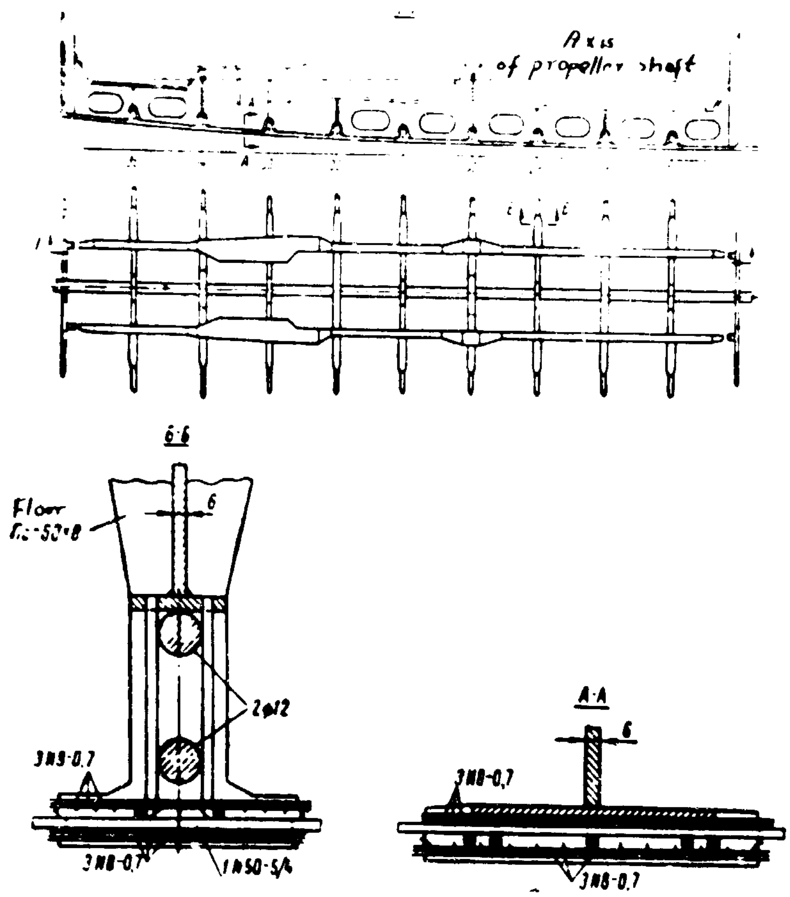

Fig.54. Diagram of Installation of Foundation Under the Main Engine.

Some examples of the attachments of the rudder device, the stern tube and the bulwark rail are shown in Figs.56,57 and 58.

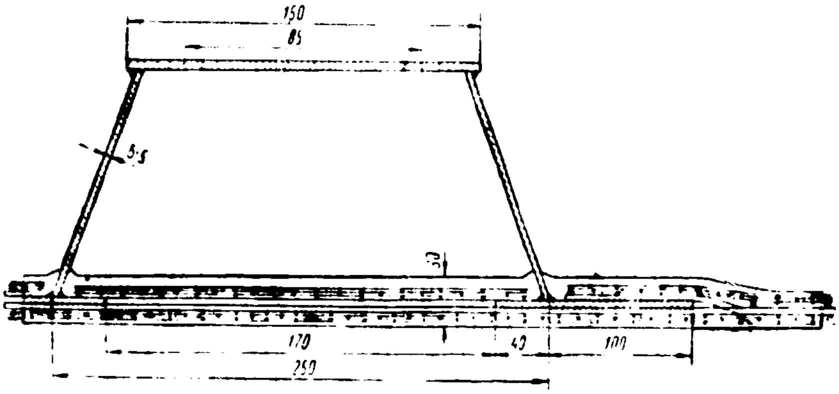

Fig.55. Installation of Foundation under the Deck Mechanisms to the Reinforced Concrete Plate.

/97

The reinforced concrete sheathing (planking) in the region of installing the rudder mechanism and the stern tube (Figs. 56 and 57) is shielded by a steel sheet, since these sections of the planking are exposed to the intensified effect of the hydrodynamic forces. A similar protection of the reinforced concrete planking with steel plates is also used in the area of installing the hawse holes (Fig.59) and the installation of the stem. The attachment of the protective steel strips to the reinforced concrete planking is achieved by their welding to the intermediate rod or additional anchoring equipment.

Examples of the attachment of the systems and pipelines to the reinforced concrete planking are shown in Fig.60.

In the case of the intersection of the planking plates by the pipelines, it is recommended that their installation be achieved in accordance with Fig.61. The installation of the bulkhead cylinders can be conducted in accordance with Fig.63. In case of the use of the design shown in Fig.63, the necessity is eliminated of the preliminary concreting of the cylinder together with the design,

and the assembly of the connection is simplified. The tightness of the joints is assured by using additional linings. Essentially, the connection in question is similar to that using the through bolts. Therefore, by drilling the reinforced concrete and utilizing the cylinders in the design indicated in Fig.62, we can simplify greatly the assembly of the systems as compared with assembly to the plates made of standard ferroconcrete, and accomplish it in analogy to the practice of steel shipbuilding.

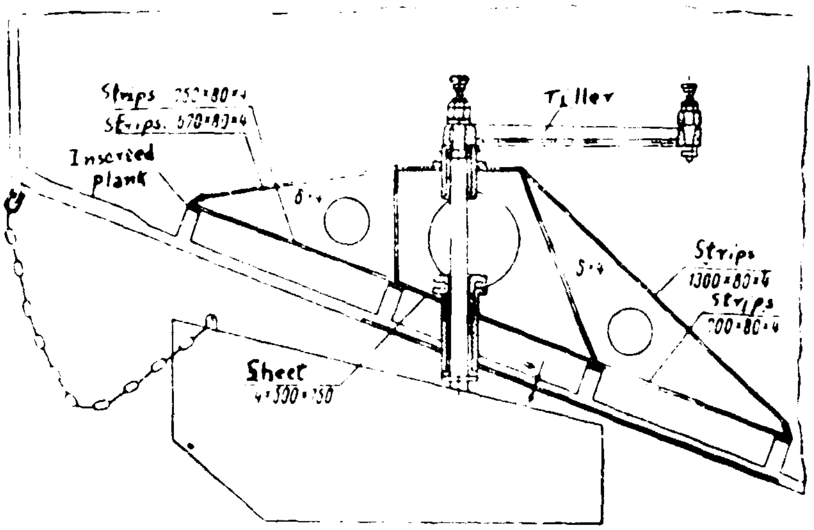

Fig.56. Attachment of Rudder Device to Hull.

/98

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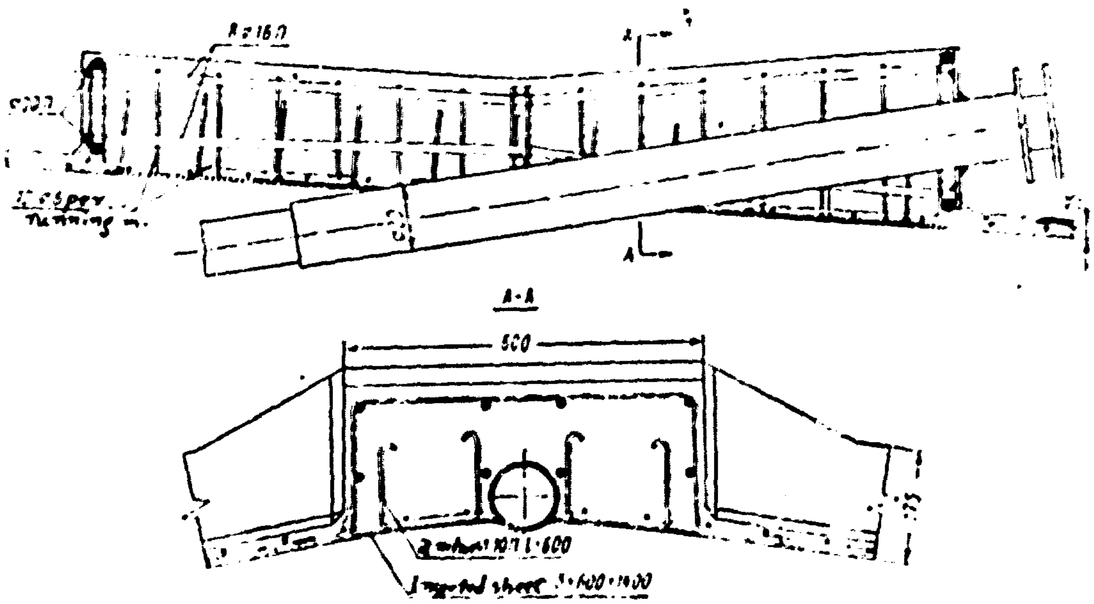

Fig.57. Attachment of Deadwood Tube to the Hull.

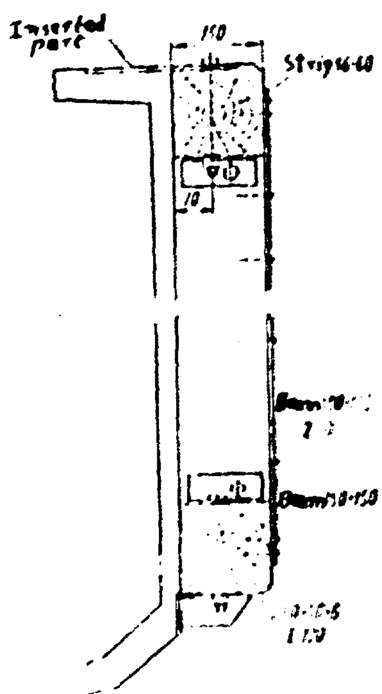

Fig. 58. Attachment of the Bulwark Rail to the Reinforced Concrete Planking.

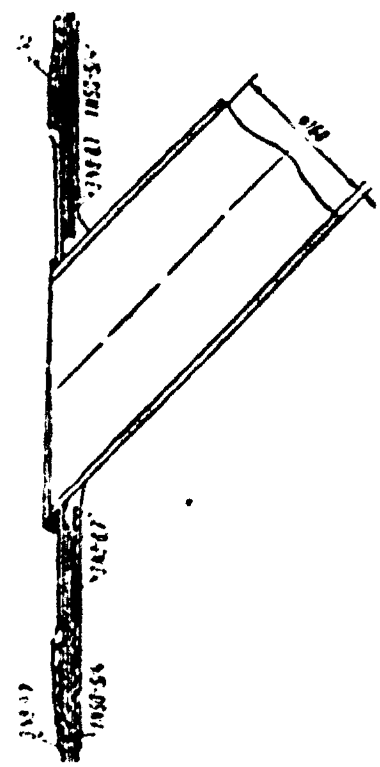

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Fig.59. Attachment of Hawse Hole to the Reinforced Concrete Planking.

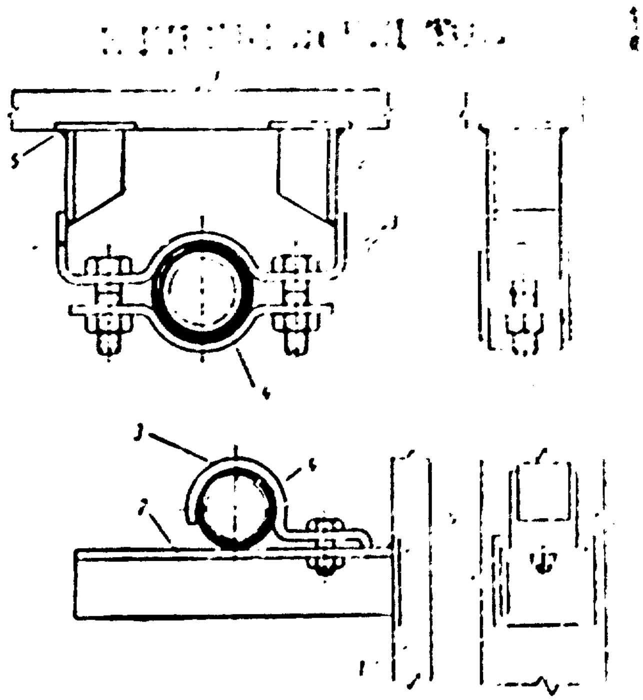

Fig.60. Examples of Fastening the Pipelines to the Plates of the Reinforced Concrete Planking. 1-reinforced concrete planking; 2-corner piece; 2-clip; 4-linings; and 5-inserted cleat. /100

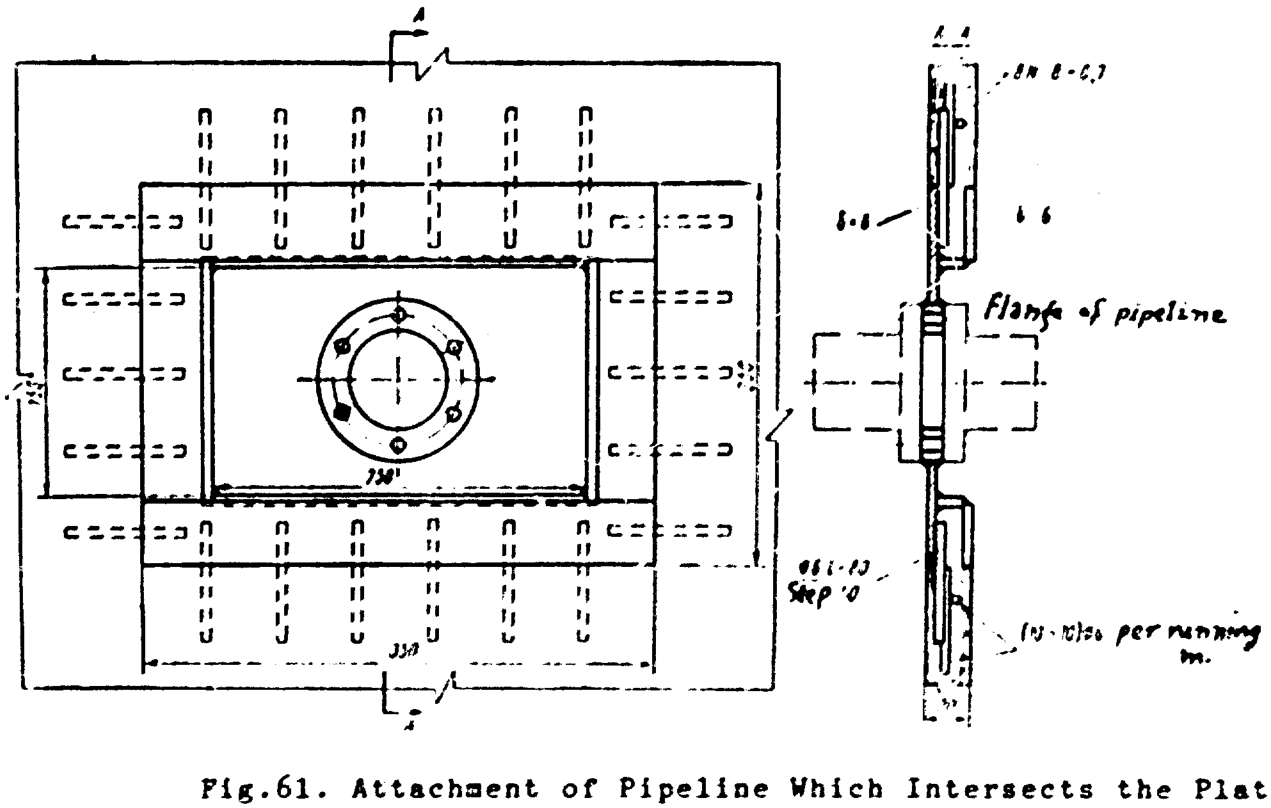

Fig.61. Attachment of Pipeline Which Intersects the Plates of Reinforced Concrete Sheathing.

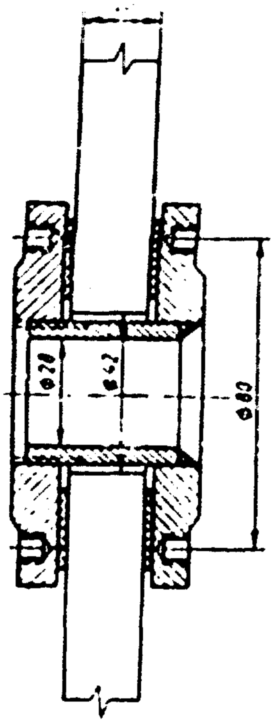

Fig.62. Attachment of Pipe-
line cylinder.

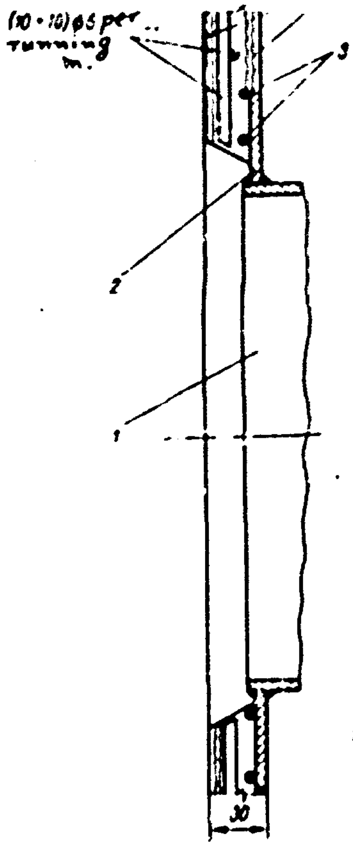

Fig. 63. Attachment of Illuminating Device to Reinforced Concrete Sheathing with Aid of Inserted Sheets; 1-illuminator frame; 2-insert -6; 3- anchoring rods 6.

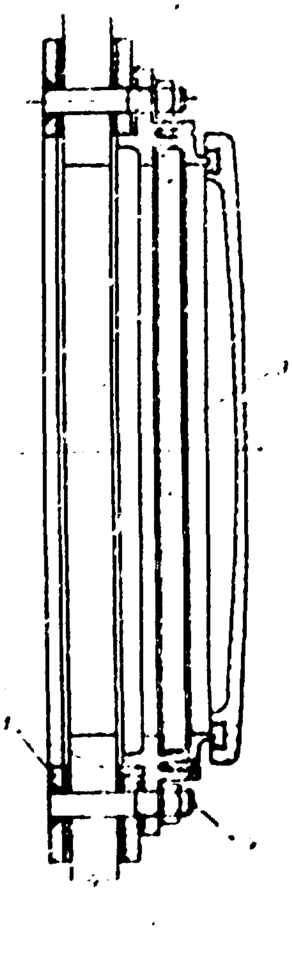

Fig.64. Fastening of Illuminating Device to Reinforced Concrete Hull Sheathing with Aid of Dowel Pins. 1- metal ring; 2-rubber linings, 3- illuminator frame; 4-dowel pin

Similarly to the attachment of the bulkhead cyliners, we also accomplish the attachment of the illuminating device to the reinforced concrete sheathing (Figs.63,64). /102

As a rule, the attachments indicated in Figs. 63 and 65, just as the fastenings using the through bolts, are utilized in the hull elements not experiencing a prolonged effect from water pressure.

Along with the bolt type of fastening, use is also made of the fastening based on inserted parts (Figs.61 and 63).

The inserted parts for the attachment of the lightly-loaded saturation elements (electric wires, lighting fixtures, etc.) are rarely utilized. In these instances, the attachment with the aid of cement-type or epoxyd glues is becoming more and more popular.

Chapter III. TECHNOLOGY OF PRODUCING THE DESIGNS AND HULLS OF SHIPS FROM REINFORCED CONCRETE

Section 16. Effect of Reinforced Concrete on Selection of Method for Building a Ship

The questions of the technology applied in the preparation of the designs and the building of ship hulls of reinforced concrete have primary importance. The relatively slight thickness of the parts, the thin protective layer, the wire mesh reinforcements and the cement-sandy concrete introduce features into the technique used in producing the reinforced concrete designs. The indicated features of the material and also the quality and care used in performing the operations determine to an even greater extent than for standard ferroconcrete the operating capability of the marine reinforced concrete designs.

The hulls of the reinforced concrete ships as well as the ferroconcrete ones can be made monolithic, sectional-monolithic and by sectional methods.

At the present time, the monolithic method is most popular. The sole exception is the building of a self-propelled driftwood hoisting floating crane with a lifting capacity of 10 tons.

The monolithic method does not require the application of extensive and complex equipment, while the construction of the reinforced concrete hull of a ship by this method can be accomplished in the building slip areas without special equipment. The positive aspects of such a method condition the economic effectiveness of its application in a case of the individual and small-scale construction of ships, and also in the building of ships of small dimensions.

The application of reinforced concrete in place of ferroconcrete simplifies the monolithic method of building ships and makes it more effective. This is /104
achieved because for making the reinforced concrete hull monolithic, in distinction from a ferroconcrete hull, cement forms are not needed. The cement-sandy concrete is retained well by a bundle of thin fine-mesh screens, in which the concrete is pressed by a worker and is rubbed simultaneously from both sides so that the operation

of making it monolithic becomes similar to a careful plastering.

If we take into account that the cost of the form during the construction of ferroconcrete ships by the monolithic method reaches 20% of the cost of the ship hull, there become quite apparent the economic advantages which are achieved with the formless production of the monolithic reinforced concrete hulls in the monolithic method of their construction. As a result of the circumstance indicated, the area of effective application of the monolithic method of building the reinforced concrete ships expands more rapidly as compared with the ferroconcrete vessels.

At the present time, we have not yet developed the means for the mechanized production of the sectional reinforced concrete ships, and the accomplishment of the intersectional connections of them is still more laborious than for the design made of ordinary ferroconcrete. Therefore, the use of the formless monolithic method of construction should be regarded as a technically and economically justified solution for the present time, in spite of the general tendency in shipbuilding to convert to the prefabricated (sectional) methods.

At the same time, the disadvantages typical of the monolithic construction method (manual labor, long duration of the cycle on the building slips, and in part the seasonal nature of the work) provide evidence that this method does not match the modern level of development of shipbuilding production. Therefore the introduction of the prefabricated method in the mass production of reinforced concrete ships and in the creation of the necessary highly-productive equipment for producing the prefabricated designs and the formation of a ship hull on the slip is a pressing modern requirement.

In the prefabricated method of construction, the division of the vessel hull into sections and the sequence of forming the hull on the slip are established by way of comparing the prime cost and the cycle of building in case of different types. We consider as efficient that division of the hull into sections and the sequence

of its formation on the slip under which we achieve the least prime cost and shortest duration of shipbuilding time.

The pattern of sequence for forming the ship hull on the slip should provide for the maximal parallel setup for conducting the hull, mechanical and electrical assembly, conduit, insulating tasks, and also the work involved in equipping and finishing the quarters. Taking into account the effect of the factors of a design nature, we can conclude that in a general case, the method of building a reinforced concrete ship should be selected in dependence on the type of ship /105 under construction, the configurations, dimensions, design features, series condition of construction, and the production-engineering conditions of the shipyard.

Section 17. Basic Features of Engineering in the Construction of Reinforced Concrete Ships

We indicated in Chapter 1 that reinforced concrete in essence is a thin-walled type of ferroconcrete, whereby there is mainly established the specifics both of the actual reinforced concrete design as well as the technology of their production.

If we consider the basic technological operations which are accomplished during the production of the reinforced concrete designs and the formation of the hulls in reinforced concrete ships, we can be convinced that basically they do not differ from the principal technological operations applied in the building of ships from standard ferroconcrete. Both in the case of utilizing reinforced concrete and in the case of using conventional ferroconcrete, the following basic work stages take place: The preparation of the reinforcing material; the preparation of the inert masses; the preparation of the reinforced frames of the prefabricated parts or of the ship hull as a whole and the concreting of the prefabricated parts, or the task of making the hull monolithic (under the monolithic building method); the heat-moisture processing of the concreted design; the assembly of the prefabricated parts on the slip and the performance of the intersectional connections; the finishing of the hull

designs; the testing of the hull for watertightness; the assembly of equipment, devices, systems, etc; the insulation and finishing of the rooms on board; the lowering of the ship into the water. At the same time, certain engineering operations in the building of ships from reinforced concrete have a number of features which must be taken into account in developing the technique of building reinforced concrete ships, along with the requirements imposed during the construction of ships made of conventional ferroconcrete.

Not dwelling on the characteristics of the engineering operations, common for the reinforced concrete and ferroconcrete ships, which are described fairly well in the literature on the technique of building ferroconcrete ships *, let us examine only the basic features of performing the operations involved in building ships of reinforced concrete.

Preparation of Reinforcing Material. The rod-type reinforcement utilized in the construction of reinforced ships is subjected to the same processing as /106
in the building of ships from standard ferroconcrete (cleaning of rust and scale, scraping, cutting, bending, etc).

The reinforcement steel, arriving in the form of rods and in coils, is straightened, cleaned and cut on the truing-cutting automatic lathes. The fiber gratings (woven meshes) during the preparation are lubricated to protect them from corrosion, and in such a form are delivered to the shipyard. The anticorrosion lubrication of the screens must be removed, since in the presence of a lubricant, the adhesion of the concrete with the mesh wires is reduced considerably.

The cleaning of the lubricant from the screens is accomplished by a thermal or chemical method.

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*K.A. Abrosimov, A.A. Mil'to, A.M. Pasinskiy. "Technology of Ferroconcrete Shipbuilding", Sudostroyeniye (Shipbuilding), Leningrad, 1965. N.M. Yegorov. "Technology of Building Ferroconcrete Ships", Rechizdat (River Publishing House), Moscow, 196[illegible].

Under the thermal (heat) method of cleaning, the screens are held in heating furnaces at a temperature of 250-300°C until the lubricant has been completely burned away.

The chemical cleaning of the screens is accomplished in washing machines, and in the absence of such, can be cleaned in any tank with a hot alkali solution of the following composition (g/l):

Caustic soda (sodium hydrate)......40-50
Sodium carbonate..................80-100
Liquid glass......................10-15

The treatment of the mesh is conducted at a solution temperature of 80-90°C with a subsequent washing in hot water at a temperature of 70-80°C. To avoid the corrosion of the screen from which the lubricant has been removed, its cleaning should be conducted directly before use. The cutting of the screens according to the dimensions of the designs being produced is conducted with cutting shears (electrical or manual).

Assembly of Reinforcing Frames. The ribbed reinforced concrete designs can have metal and ferroconcrete ribs (beams of the set).

The process of producing the reinforced frames include:

- preparation of the reinforced parts (of the bundles of woven screens, frames of ribs and supports);

- assembly and welding of the reinforced parts in the spatial designs, i.e. the flat or curved reinforcing sections; and

- preparation of reinforcement for the intersectional joints and monolithic parts.

The assembly of the reinforcing frames of the ferroconcrete beams of a set, of the intermediate screens of the reinforced concrete plates and of various supports in the production of the ribbed reinforced concrete designs does not differ in any way from the assembly of the reinforced designs made of ordinary ferroconcrete.

However, the assembly of the reinforced frames of the plates has certain features. As is known, the reinforced frame of a plate consists of several layers of thin meshes and can have an intermediate rod-type grating. /107

In the production of the elements of the reinforced concrete designs, the thin wire screens are usually assembled into bundles, in which the screens are interconnected with the aid of binding wire or by spot welding. The number of such joints must provide a sufficiently smooth surface of the bundle (without bulges) and the required assembling connection of the screens with each other.

In the case of the combined reinforcement, the reinforced frame of a plate is assembled from two bundles of fine screen, with a number of layers prescribed according to the plan, and with an intermediate rod-type grating (for welding or for wire twisting). First, on one of the bundles, we lay the intermediate rod-type screen, which is attached to it with binding wire (in twists) or by welding; then the second bundle of screens is installed; it is also attached to the intermediate rod-type screen.

The welding of the wire screens between each other by lap-welding and in bundles is conducted with the universal spot-welding tool UTP-3, with the application of copper laminated electrodes 30 X 60 mm, according to the conditions (approximately):

Strength of welding current........ 3600 amps
Secondary voltage.................. 11.5 volts
Welding time....................... 0.05 sec

The sets of wire screens are welded with the intermediate rod-type screens and with the reinforced frames of the ribs also by the universal welding tool UTP-3 with the application of copper electrodes 5 mm in diameter roughly according to the conditions:

Force of welding current........... 8200 amps
Secondary voltage.................. 11.5 volts
Welding time....................... 0.05 sec

The rods 8 mm in diameter and up are connected by vat-type butt electric welding with semiautomatic devices using alloyed wire or in a carbon dioxide atmosphere. The rods with a diameter less than 8 mm are welded together by the seam lap welding using semiautomatic devices.

In the reinforced frames of the sections, we install all of the inserted parts, not extending beyond the limits of the plate's thickness. The inserted parts are fastened to the reinforced frames by welding with semiautomatic devices in a carbon dioxide atmosphere (or by alloy wire) to the intermediate screen, the frames of the ribs and supports, or to the specially installed anchoring rods.

The electric welding by alloy wire and the welding in a carbon dioxide atmosphere are performed in the same way as during the assembly of the reinforced frames of the ship designs made of standard ferroconcrete. /108

The various slots and openings in the reinforced frames for the hatches, vents, lights, compartments, etc. are made according to the marking done with a gas-type cutter.

In the case of the monolithic method of construction, the fine meshes in the distribution over the outlines of the ship are stretched to eliminate sagging. In this connection, the reinforced designs of the monolithic hull elements can be assembled on the slipway from the prefabricated reinforced sections.

Preparation of Concrete and the Concreting of Designs. The preparation of the cement-sandy concrete is conducted in the mortar mixers or in the concrete mixers with forced agitation. The higher requirements imposed on the mixing of the cement-sandy concretes as compared with the standard concretes are caused on the one hand

by the relatively low water-cement ratio for these concretes, and on the other hand by the large specific surface of the inert (sandy) filler. In connection with this, we require a more careful and extended agitation of the mixture for reducing it to a uniform consistency.

In order to provide the careful mixing of the concrete batch, the materials are loaded into the concrete mixer in the following order: First, we add the cement and sand,and then the dissolving water. The duration of the mixing is not less than 2-3 minutes, and is established finally on the basis of the smoothness of the concrete mix, established visually.

The monitoring of the quality of preparing the cement-sandy concretes includes: the checking of the quality of the components (cement, sand,water and additives), the control of the accuracy of batching the components and the verification of the convenient handling of the prepared mixture. From the listed control operations,the primary one is the checking of the handling convenience of the concrete mixture, which is established not based on the settling of a standard cone but with a technical viscosimeter, wherein the index of placement, measured in seconds, is established depending on the quantity and number of the wire screens, and in each actual case is specified on the basis of experimental checking. For the reinforcement used in the reinforced concrete ship designs, the placement index comprises 15-25 seconds.

The selection of the composition of concrete mixture can be conducted by any method, under the condition of complying with the

prescribed requirements for the properties of the concrete mixture and of the hardened concrete.

The reinforced concrete sections can be prepared both by the stand and by the assembly-line methods. The feasibility of using any given method of producing the sections depends on the type, dimensions and weight of the section, and in each actual case is established by a technical-economic calculation. /109

The basic technological sequence of producing the reinforced concrete sections in case of the bench or assembly-line methods is identical and includes:

- preparation of forms;
- placement of the reinforcing frames in the forms, and the installation of falsework for attaching the outlets of the wire meshes and the reinforcing rods;
- the placement and packing of the concrete;
- the heat-moisture processing (steaming) of the concreted sections;
- the stripping of the sections and their conveyance to a warehouse or a slipway; and
- the treatment of the connecting edges of the sections.

In case of the mechanized placement, packing and smoothing of the concrete mixture with vibrating-shaping units, the flat ribbed sections are concreted in the metal form-matrices below in the form of ribs; the ribless sections are concreted on the flat metal stands.

The sections of curvilinear shape are made in wooden forms, trimmed in order to increase their turning ability and to improve the quality of the section's surface with water resistant plywood or with thin-sheet steel. Such sections can be concreted both below and above, with ribs.

The placement of the concrete in the sections (curvilinear and flat, having reinforcements), made upward in the form of ribs, is accomplished manually during the packing of the concrete with vibrating rods, with sectional and deep electric vibrators.

For the production of the sections(especially of the curvilinear ones), and also of the monolithic hull elements, we can apply the method of the formless concreting. In this case, the concrete mixture is applied by hand to the reinforcing frame or by guniting (by spraying) with the aid of compressed air. During the period of the concrete's setting, the surfaces of the concreted designs are rubbed with dry cement. The framing beams are concreted similarly to the monolithic ferroconcrete designs with the application of falsework.

An important moment in the preparation of the hull designs from reincorced concrete is the provision of the necessary protective layer, uniform over the entire surface of the design. This requirement is quite significant, since a reduction in the protective layer to a value of less then 2-3mm does not guarantee the required corrosion resistance of the designs, while its increase above 3 mm leads to a decrease in the crack resistance.

Under the conditions, when the methods of mechanized production of the ship reinforced concrete designs have not yet been worked out, the attachment of the required protective layer is achieved with

the aid of the reinforcing rod-linings with a diameter equalling the value of the protective layer. The rod-linings are placed in /110 the form-matrix and on the bundle of screens before the concreting of the designs with a step (spacing) of 150-200 mm and after the packing of the concrete they are removed. The grooves which are caused thereby are filled by an additional brief vibration of the designs in the location of the linings.

Heat-Moisture Treatment of the Concreted Design. The aging of the concrete of the reinforced cement designs can occur under naturally moist conditions or during a heat-moisture treatment of it.

The thermal-moisture processing of such designs, in distinction from those made of standard ferroconcrete, is conducted according to the "soft" conditions with approximate parameters: the gradual heating to a temperature of 60-70°C with a rise in temperature of 10-15° per hour, isothermal heating at maximal temperature of 60-70°C for two-three hours, then a cooling at a rate of not more than 15-20° per hour.

Assembly of Hull on the Slipway. A feature of the technology of the slipway construction of the ship hulls made of reinforced concrete is the conduct of the intersectional joints and of the monolithic elements.

The reinforced concrete sections are interconnected and are also joined with the monolithic elements of the hull by way of by-passing the reinforcing outlets (of the wire meshes and of the reinforcing rods), and also are connected to the assembly inserted parts.

The fitting outlets of the sections are connected by welding in the following technological sequence:

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Table 12

Recommended Compositions of Anticorrosion Coatings for Reinforced Concrete Vessel Designs

Nomenclature and composition of coating (paint)	Content of components (by weight, %)	Number of layers	Approximate usage in application of 1 layer per sq m, g: With brush	By spraying	Remarks
Bituminous mastics:	—	[illegible]			Prepared at place of use
petroleum bitumen, grade V (GOST 6617-56)	[illegible]				Base—solution of bitumen in gasoline (1:3)
solvent-naphtha (GOST 1928-58)	[illegible]				
transformer, solar or green oil	[illegible]				
rubber cement (commercial)	[illegible]				
filler: asbestos, slag or cement	[illegible]				
Bituminous-ethanol lacquer:					Prepared at place of use
composition I (without filler)					
petroleum bitumen, grade IV-V (GOST 6617-56)	[illegible]	[illegible]	[illegible]	[illegible]	Base—bituminous-ethanol lacquer (composition 1:10)
ethanol lacquer (TU 1267-54K MKhP)	[illegible]				
composition II (with filler)					
petroleum asphalt	[illegible]	[illegible]	[illegible]	[illegible]	
ethanol lacquer	[illegible]				
filler: asbestos diabase or andesitic powder, etc.	[illegible]				
composition III (GOST 1347-41)					
bituminous lacquer No.411	10 50	2 3	80 90	10[illegible] 120	Base—bituminous-ethanol lacquer (composition 1:10)
ethanol lacquer (TU 1267-54k MKhP)	[illegible] 50				
Ethanol paints: composition I		3 4	100	150 200	Prepared at place of use
ethanol lacquer	[illegible]				Base—ethanol lacquer
asbestos (GOST 7-51)	[illegible]				
titanium white or diabasic powder	7.5				
composition II					
ethanol lacquer	80 85				
aluminum powder (GOST 5494-50)	20 15				
composition III					
ethanol lacquer	60 [illegible]				
iron oxide (GOST 8866-58)	10 [illegible]				
composition IV					
ethanol lacquer	75 [illegible]				
graphite	[illegible]				
Epoxy enamels: EP-51 or EP-56 (VTU NU 484-57)		3		200 [illegible]	Delivered in finished form Base E-4021 (VTU NU 493-57)

- the welding of the outlets of the distributing reinforcing mesh with a semiautomatic device in an atmosphere of carbon dioxide or alloyed wire;

- the spot-welding of the wire meshes to the outlets of the distributing mesh with the welding tool, type UTP-3;

- the tank welding of the outlets of the framing beams' fittings by semiautomatic welding devices in a shielded arc of carbon dioxide or of alloyed wire.

The inserted parts are also welded with the semiautomatic device.

For falsework, use is made of the metal or wooden-metal stock panels.

The intersectional joints are concreted with sandy cement of the same grade and composition as the concrete in the sections, but having a high mobility of the concrete mixture (roughly 10-15 seconds).

The concrete mixture is delivered and placed in the connections with mortar pumps. The packing of the concrete in the vertical joints with the two-sided falsework is conducted during an intensive vibration with electric vibrators fastened on the falsework; the concrete in the horizontal joints is packed with the area-type electric vibrators. /113

Attachment of the Inside Equipment of the Ship to the Reinforced Concrete Designs. The reinforced concrete can easily be drilled, and this property of it is utilized to a full extent in the accomplishment of work on attaching the internal equipment of the ship to the reinforced concrete sheathing.

To the designs not experiencing the prolonged effect of water

pressure (bulkheads, partitions, platforms and decks), the attachment of the inside fittings can be done with bolts. In this connection, the holes of the required diameter are drilled directly on the spot. Such a method of attaching the internal fittings of the ship to the reinforced concrete sheathing is much simpler and more economic than the attachment on the inserted parts.

Application of Anticorrosion Coatings to the Reinforced Concrete Sheathing of the Hull. The standard shipbuilding ferroconcrete produced with high quality is a sufficiently long lasting and corrosion-proof material, which is confirmed by the many years' practice of operating the various facilities made of it, including the ships and docks. However, the corrosion resistance of the reinforced concrete sheathing of the hull, as a result of the low value and the unreliability of the available means for controlling the thicknesses and density of the protective layer of concrete, can not be guaranteed for the full service life of the floating facilities in view of the difficulties associated with providing all the technological requirements during the production of the designs.

In connection with this, for providing the operational reliability and the long life of the ships, to the external sheathing of the reinforced concrete hull to the level of the main deck, we recommend the application of anticorrosive protective coatings.

The compositions of the recommended anticorrosive coatings for application to the reinforced concrete sheathing of ship hulls are indicated in Table 12.

Before the application of the protective coatings, the concrete surface is cleaned of contamination and incrustations of corrosion

with the aid of wire brushes or other tools, and then it is cleaned with compressed air. If on the concrete surface, we find large flaws, cracks and dents, they are repaired by rubbing with a cement--sandy solution in the ratio of 1:2 (by weight). The moistness of the surface layer of concrete prepared for the application of coatings should not exceed 5-6%. The moistness of the concrete surface is determined by drying to a constant weight (at a temperature of 100-150°C) of samples taken from the concrete surface in two-three places at the depth of the protective layer. The percentage of moistness is established with the formula

$$A = a-b/a \cdot 100,$$

/114

where A = the moistness of concrete;

a = the weight of sample prior to drying;

b = weight of sample after drying.

The protective coatings are applied to the concrete surface on a previously applied base.

As a base, we use:

- for composition I of the protective coating- a solution of bitumen in gasoline of composition 1:3 (by weight);
- for composition II - bitumen-ethanol lacquer with a composition of 1:10;
- for composition III- ethanol lacquer; and
- for composition IV - base E-4021 (VTU KU 496-57).

The base is applied by hand or mechanically with the aid of compressor-type or compressorless sprayers, in a regular thin layer, without leaving uncovered spots or inflows.

The working consistency of the base (viscosity) in the paint sprayer operating under a pressure of 3-3.5 atm should be within the limits of 18-22 seconds based on the VZ-4 viscosimeter at a temperature of 20°C. The working viscosity of the bases is attained by diluting them with the solvents:

- for mixture I - with gasoline;
- for mixtures II and III, with ethanol lacquer; and
- for mixture IV, with the R-40 solvent or acetone.

For the development of a strong coating, the base should be well dried. An insufficiently dried film of the base under the effect of the solvents, forming the lacquers and the paints, can dissolve, as a result of which bubbles and wrinkles are formed. The normal length of drying the bases for the recommended protective coatings is 24 hours at an ambient air temperature of 18-23°C. The painting with a cold bitumen composition (mixture I) is conducted at a composition temperature not below 60°C and at an ambient air temperature not below 8°C. The composition is applied to the protected surface with a brush and is smoothed out with a wide spatula.

The paints, lacquers and enamels (mixtures II,III and IV) are applied to the concrete surfaces (already provided with a base) with paint sprayers. Under the necessity of performing the work with a brush, the paints, lacquers and enamels are thinned to a working viscosity with the appropriate solvents.

The painting of the outer surfaces (for all mixtures of the coatings) is conducted at a temperature not below 8°C; the performance of the painting tasks during rainy weather is not permitted. After the application of each layer of paint or enamel, we conduct a drying

of the coating in accordance with the technical conditions for this material. After the application of the specified number of layers, the coating is subjected to a final drying fot two-three days. /115

Section 18. Examples of Construction of Reinforced Concrete Ship by the Monolithic Method

The domestic and foreign practice has fairly extensive experience in the monolithic construction of the hulls of reinforced concrete ships.

In Italy, in 1943, at a shipyard in Venice, the construction was started on a motorship with a deadweight of 400 tons and of three self-propelled transports with a deadweight of 150 tons each. However in connection with the military operations developing in this area, the construction of the ships was interrupted. After the end of the war, the construction of the reinforced-concrete ships in Italy was renewed.

In 1945, at a shipyard in Anzio, a motor-sailing vessel "Irene" with a deadweight of 165 tons was built. In the later years (1945-1948), in various regions in Italy, there were built from reinforced concrete launches, yachts, pontoons, and also transport, fishing and other ships.

The sailboat "Irene" was built by the shipyard workers at the yard in Anzio destroyed by the war, at which no mechanical or electrical supply was operating. For its construction, there could be used only the slipway and the launching platform (track).

Specifications of the Ship

Length between perpendiculars, m	21.6
Maximum length, m	6/24

Molded depth amidships, m............ 3.2

Power of main engine,h.p.............316

The hull of the sailboat is of ribbed design with a transverse framing system. The lines of the hull are the conventional curvilinear ones.

The design feature of the hull consisted in the fact that the framing beams were ferroconcrete; the sheathing and the deck flooring were of reinforced concrete. The framing beams with a width of 60-80mm had a height ranging from 250 mm (for the side branches of the frames, beams and carlings) to 400 mm (for the floors and keelsons). The stronger ferroconcrete beams were located in the area of the keel, the bow and the deck stringer. The outer sheathing made of reinforced concrete was 35 mm in thickness.

The construction of the ship was accomplished in the open slipway area beneath the keel. Initially on the slipway, we assembled the skeleton of the hull's framing beams (both the longitudinal and the transverse ones). All the framing beams had reinforcement typical for the standard ferroconcrete ships. The assembled reinforced framework comprised a fairly rigid spatial (three dimensional) design, reproducing the form and outlines of the ship under construction. Then from the outside of the hull (with the exception of the deck), on the reinforced carcass of the framing, one upon the other we placed in succession the following: four /116 layers of fine wire mesh, weighing one kg per square meter, three tiers of reinforced rods with a diameter of 6 mm (two tiers were placed in a longitudinal and one in a transverse direction) and once again, four layers of wire mesh. The three-layered reinforced design

of the hull sheathing obtained in such a manner was connected with steel binding wire.

Fig.65. The Reinforced Concrete Yacht "Irene" During the Construction Period.

The reinforced carcasses of the framing beams were connected with the reinforced concrete sheathing in the process of its reinforcement by way of bending the outlets of the yokes around the rods, located in the sheathing between the mesh bundles. The transverse bulkheads, the platforms and partitions within the hull were reinforced simultaneously with the reinforcement of the outer sheathing. All of the reinforcing tasks were done by hand.

Upon the completion of reinforcing the hull elements (with the exception of the deck, superstructure and deck-house) we conducted their concreting,without falsework, continuously from the bottom to the deck. For the concreting, we used sandy concrete of plastic consistency with the output of 1,000 kg of cement per cubic meter of concrete. In the concreting of the reinforced concrete elements, the concrete was applied to the reinforced framework by hand from within the hull, and was pressed through the screens; at the same

time, from outside, we accomplished the smoothing and finishing of the concrete surface (Fig.65). The deck and also the stern superstructure with the deck-house were reinforced, and then were also concreted after completing the concreting of the hull elements lying below. /117

To the finished hull, there were applied the anticorrosive and anti-fouling coatings.*

The reinforced-concrete pleasure yacht "Nennele" was built in Rome in 1948.

Specifications of the Yacht

Maximum length, m	12.5
Length along the design water line m, ...	9.5
Maximum width, m	3.2

From a design standpoint, the hull of the yacht "Nennele" differs from the hull of the sailboat "Irene" chiefly by virtue of the fact that in place of the ferroconcrete beams, as framing use was made of the steel inch pipes. The yacht had a reinforced concrete sheathing with a thickness of 12 mm.

In the construction of the hulls, the framework was first assembled from steel pipes. Then, the sheathing of the hull and the transverse bulkheads were reinforced. The reinforcement of the outer sheathing consisted of seven layers of wire mesh and of one tier of longitudinal rods with a diameter of 6 mm, set 50 mm apart. The longitudinal rods divided the mesh reinforcement into two bundles (three layers of mesh

* The composition of the coatings is not given in the literature.

within and four layers outside of the hull).

The hull was concreted from inside; the sandy concrete was applied manually to the meshes and was smoothed out from the outside. After the completion of construction, the yacht was hauled on a truck to Anzio and launched into the water.

Subsequently, the basic engineering principles used in building the yachts "Irene" and"Nennele" were subsequently applied in building the small reinforced concrete vessels in other countries, especially in the building of the pleasure cutters and the commercial ships with a length ranging from 7.3 to 16.7 m in England and New Zealand.

The monolithic method was also applied in building the reinforced concrete yachts and launches in our country. In this connection, along ith the formless preparation of the hulls, in a number of cases, in the concreting, falsework was used, which generally speaking is inefficient, but evidently this took place because of the insufficient experience.

An example of producing a reinforced concrete hull with the application of a cement form (mold) can be provided by the building of the yacht "Tsemental". The basic difference in the tecnhique used in building the yacht "Tsemental" from the method used in building the "Irene" was as follows.

First, the hull of the yacht was built upward from the keel on a previously prepared form-mold, in the capacity of which use was made of the hull of a wooden yacht, also installed with the keel upward. The reinforcing framework (placed in the mold) of the hull sheathing, consisting of five layers of wire mesh No.10, was concreted continuously from above downward, i.e., first the keel and then the side /118 were concreted. After the attainment by the concrete of the

necessary strength, the reinforced concrete shell was removed from the form, placed on the slipway with the keel downward, and the finishing (trimming) of the internal surface of the reinforced concrete sheathing was conducted.

Secondly, the hull framing was installed after the preparation of the reinforced concrete sheathing (of the hull shell). The previously bent frames (ribs) made of steel pipes were fitted in place within the hull and attached to the reinforced concrete sheathing by wire splices which were passed through the small openings in the sheathing, punctured from both sides of the frames through each 120-180 mm and subsequently made monolithic. After the attachment of all the ribs, the installation of the beams of the deck and superstructure was conducted; the transverse bulkheads, the deck and the superstructure were reinforced. The concreting of the deck, the transverse bulkheads and the superstructure was achieved without the use of a form (mold).

In the period from 1957-1964, a group of sailboat enthusiasts built the reinforced concrete yachts "Opyt" (Experience), "Progress" and "Mechta" (Dream). The hulls of the yachts were made with the keel upward, in which the basic difference from the technique of building the "Tsemental" consisted in the fact that in the place of the form, use was made of the framework assembled on the slipway from the duplicated outlines of the hull of the wooden templates, which were interconnected by wooden rods. Along the aligned framework, we conducted the reinforcement of the hull, consisting of rods mainly 6 mm in diameter, and of 6 layers of metal wire mesh.

The hull was concreted without the use of a form, by way of applying a cement-sandy concrete made of a mesh reinforced design.

By the monolithic method, without the use of falsework or wooden molds, we built the reinforced concrete launch "Energostroitel'" (Power Builder) which was launched in autumn of 1960. The reinforcement of the hull framing of the launch was achieved with rods of a smooth profile, having a diameter of 14 and 10 mm; the reinforcement of the sheathing was done with five layers of wire mesh, with a mesh size of 10 X 10 mm, and wire 0.5 mm in diameter. All of the framing units were assembled on the slipway in the framework in position with the keel downward, and were connected by electric welding. Prior to the reinforcement of the sheathing, the reinforcing framework was taken from the slipway and turned into position with the keel upward.

The concreting of the hull was accomplished with two methods. First, the concrete was applied from outside, and to speed up the hardening of the concrete, the hull was subjected to steaming out in a chamber, and then in the same place, the hull was turned with the keel downward, whereupon it was concreted from inside and again steamed out.

The examples which have been reviewed of the monolithic construction of small reinforced concrete ships permit us to confirm the finding previously made to the effect that from the technological and economic standpoint, the most improved is the monolithic method including the use of the formless concreting in one step, which is achieved fairly simply in the construction of a ship in a position with the keel downward. Such a construction method, requiring a

twofold concreting of the hull, can also be effective for the small vessels (such as the sloops and small yachts), especially if as a mold (form), we can utilize the hull of a ship which has gone out of commission.

Section 19. Examples of Building Reinforced Concrete Ships by the Prefabricated and Prefabricated-Monolithic Methods

The only example at the present time of the prefabricated method of building a ship with a hull and superstructure of reinforced concrete is the self-propelled pontoon hoisting crane with a lifting capacity of 10 tons, built in our country in 1964.

The designs of the hull and superstructure of the pontoon crane, developed with consideration of their production completely by the prefabricated method, are reviewed in Chapter 2.

The hull is assembled from 45 plane and 2 curvilinear ribbed sections; the superstructure is made of 15 plane sections. The maximal dimensions of the sections comprise 11.3 X 2.1 m; the weight is up to 2.5 tons.

The hull sections were interconnected mainly by welding the reinforcement outlets and the inserted metal parts, with a later monolithizing of the intersectional joints with sandy cement (Fig. 66).

The intersectional joints and the connecting edges of the plates of the sections had a thickening up to 50 mm owing to the reinforcement of the contact zone and the adjacent edge-plates by the welded frameworks made of rods with diameters of 6 and 10 mm. In this manner, the design of connecting the reinforced concrete sections in the given instance does not differ appreciably from

that applied in building the ship hulls of conventional ferro-concrete. The sections of the superstructure were interconnected by welding the inserted parts, and by mounting on bolts.

The attachment to the reinforced concrete designs of the hull and superstructure, of the various mechanisms, devices, systems and rigging was achieved with the aid of the inserted parts, which were mounted in the sections and the intersectional joints prior to the concreting.

The construction of the pontoon hoisting crane was conducted in the Sokol shipyard of the MRF, which had neither special engineering equipment and rigging for the mechanized production of prefabricated concrete reinforced elements, nor experience in building such ships.

In addition to the equipment available at the shipyard, there were prepared only a jig for the assembly of the reinforcing frameworks of the sections, a stand for concreting the flat rein- /120 forced concrete sections, and a mold for reinforcing and concreting the curved sections. The reinforcing frameworks of the flat sections were assembled with ribs downward in a special jig made of previously prepared distributing screens, of welded frameworks of ribs and reinforcements, and also of metal wire meshes. Initially, in the jig we installed the frameworks of the ribs, placed two or three layers of wire mesh, the distributing reinforcement screen and the reinforcement frameworks; then we bent away the yokes of the frameworks of the ribs and installed the remaining two-three layers of wire mesh.

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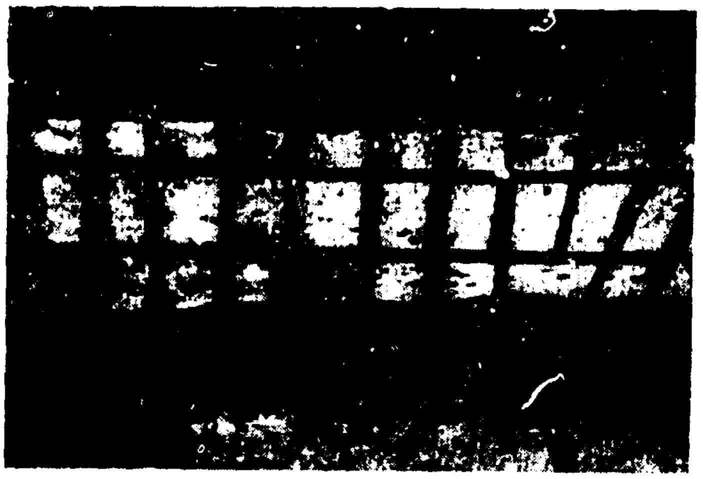

Fig.66. The Joining of the Reinforced Sections using the Welding of Reinforced Outlets of the Intermediate Screen.

The framework was finally fastened with wire splices. In the reinforcing framework of the sections, with the aid of gas cutting, we made holes and slots, installed all the inserted parts: the metal strips, sheets, the hatch coamings, etc. (Fig.67).

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Fig.67. Reinforcing Framework of the Deck Sections, with Installed Inserted Parts.

The finished reinforced frameworks of the flat sections were transported with a crane with the aid of a special traverse, and were placed ribs upward on a flat metal stand (Fig.68).

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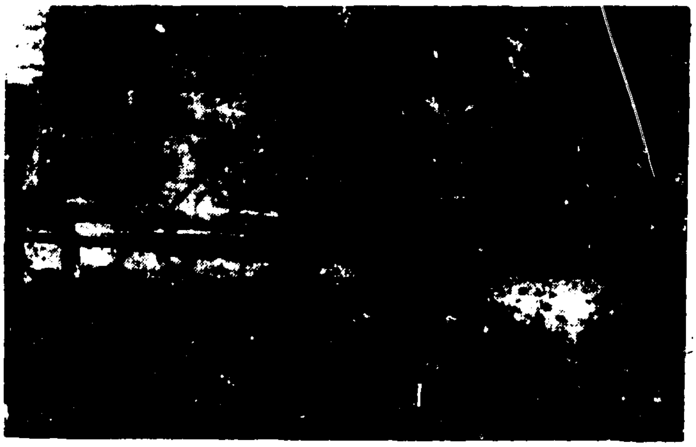


Fig. 68. Placing the Reinforced Framework of the Sections on Stand for Concreting.

The reinforcing frameworks of the curved sections were assembled with ribs upward on a wooden form, duplicating the lines of the curved part of the ship hull. The concreting of the curved sections was also conducted directly on this same form.

Before the concreting in the reinforced framework of the section, we installed a wooden falsework of ribs, and also a shaped form, limiting the dimensions of the section which was being produced (Fig.69).The sections were concreted in position with the ribs upward. The cement mixture was compacted with the area-type and deep electro-vibrators. The necessary protective layer on the side of the form stand was fixed with the aid of the reinforcement rod-linings 3 mm in diameter which were inserted between the stand and the reinforcing section, and were removed after the packing of the cement. /123

After the concreting, to speed up the aging of the concrete, the sections were covered with tarpaulin and were steamed out

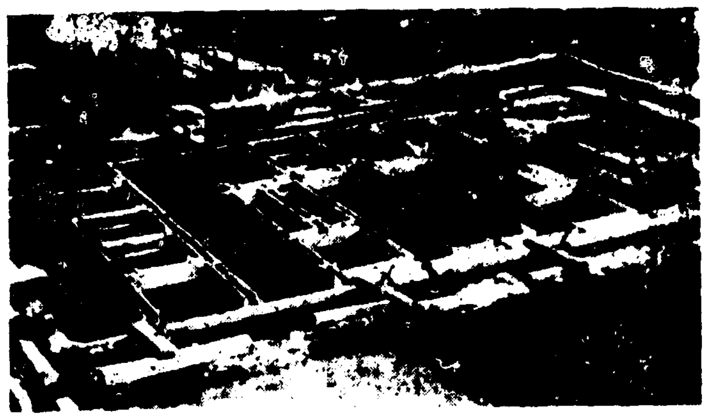


Fig. 69. Reinforced Concrete Section Prepared for Concreting.

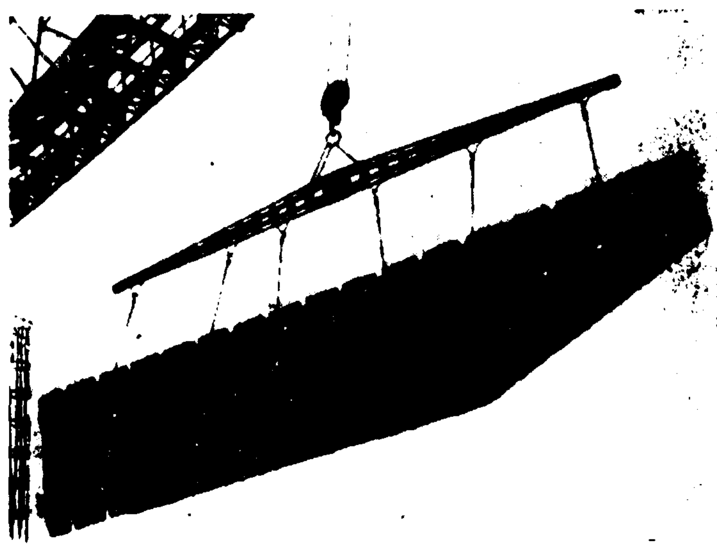


Fig. 70. Transporting the Side Section with a Crane.

directly on the forming stands.

The stripping and the removal of the sections from the stands were conducted after the attainment by the concrete of a compression strength of 250-300 kg/cm^2.

The removal of the concreted sections and their hauling

to the supply area and to the slipway were performed by a crane with a lifting capacity of 3 tons, equipped with a special traverse (Fig.70).

Fig.71. Assembling the Sections of the Stern Transom of the Pontoon Hoisting Crane.

Prior to the assembly, all of the connecting edges of the sections were subjected to a mechanical cutting.

The ship hull was assembled on the slipway which consisted of four rows (lengthwise of the ship) of keel blocks, assembled from wooden beams, with transverse wooden supports placed on them. Prior to the assembly of the hull sections, the slipway was leveled and on its face the control and base lines were drawn.

The assembly of the hull started with the installation of the curvilinear section of the bottom in the area of the engine room.

The sections were mounted on the slipway with the aid of a crane having a lifting capacity of 3 tons (Fig.71).

The hull was shaped in such a way that after the assembly of the vertical sections, the closed compartments were formed:

initially, we installed the bottom sections, then the side sections, the transverse and longitudinal bulkheads and finally the deck sections (Fig.72). After the installation and fastening of the sections the slipway in the planned position, the operations were performed on the straightening and welding of the reinforcing outlets of the sections, and also the reinforcement of the inter-sectional joints. The reinforcement outlets of the reinforced concrete plates and the assembly inserted parts were welded by manual electric-arc welding. The electric-arc welding was applied in joining the outlets of the reinforcement of the framing beams. /124

Fig. 72. Assembling the Hull of a Pontoon Hoisting Crane on the Slipway.

In the capacity of a concrete form for the connections of the sections, we used the wooden panels, which were installed after the completion of the welding operations and the placement of the inserted parts in the joints. The connecting points of the vertical hull units had a two-sided falsework.

The sections' seams were made monolithic with cement-sandy concrete of the same composition as the concrete in the sections. The concrete mixture was placed by hand and was compacted with deep electric vibrators. The aging of the concrete in the seams occurred under natural conditions. The joints of the bottom sections, and also of all the vertical sections, with the bottom and with each other, were concreted prior to the installation of the deck sections. At the attainment by the concrete of a compression strength of 300 kg/cm^2, the joints were struck, then we trimmed (finished) the outer and inner surfaces of the ship hull. The control of the concrete strength was accomplished by compression tests of the control sample cubes with sizes of 7 X 7 X 7 cm, made of concrete, which was used for concreting the hull units, and was maintained under the conditions of their hardening (setting).

The reinforced concrete superstructure was mounted on the finished hull made of flat ribbed sections. The sections of the outer panels of the superstructure were connected with the hull by way of welding the inserted parts; they were connected with each other with bolts. The cover of the superstructure was connected /125
with the wall panels with inserted parts. The cover sections were interconnected on reinforcement outlets. The intersectional joints of the superstructure were finished with sandy cement. The internal bulkheads and partitions of the superstructure were made of wood.

Prior to launching, the hull of the pontoon-type crane was tested for strength and watertightness (Fig.73). After the conduct of the tests, by means of lifting devices, the hull was placed on

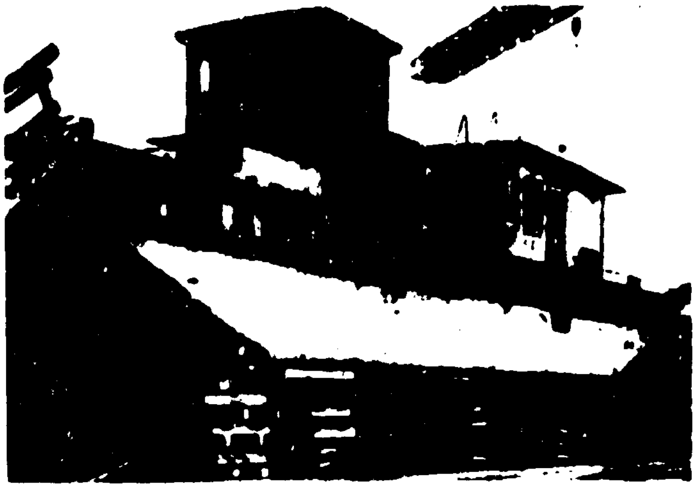


Fig. 73. Hull of the Pontoon-type Hoisting Crane during the Strength Tests.

a balanced launching device. The launching of the hull into the water was accomplished along transverse slides in a sloped position.

All of the finishing assembly operations on the ship were conducted by a specialized shipbuilding firm, which was engaged with the building of the same cranes, but on a steel hull. The greater part of the finishing operations were performed while afloat, but for the installation of the propulsion-steering unit of the ship and of the crane derrick, the hull was raised on the slip.

The assembly of the mechanisms, devices, systems and various onboard equipment was accomplished by the same methods as in the building of the steel pontoon-type hoisting cranes. The basic difference consisted only in the fact that the foundations under the mechanisms, the equipment etc, were installed and attached to the metal inserted parts, concreted in the reinforced-concrete designs, either with the aid of electric welding or by the use of

bolted joints. /126

With the raising of the ship hull on the slipway, it became possible to inspect its underwater section after a half year stay of the hull in water, as a result of which we noted the pitting of the wire meshes in the places where the protective layer was less than 2 mm. Therefore, it was decided to apply an anticorrosive coating to the underwater part and the zone of variable level of the hull. As such a coating, we chose the EKS-5 paint with a base of ethanol lacquer.

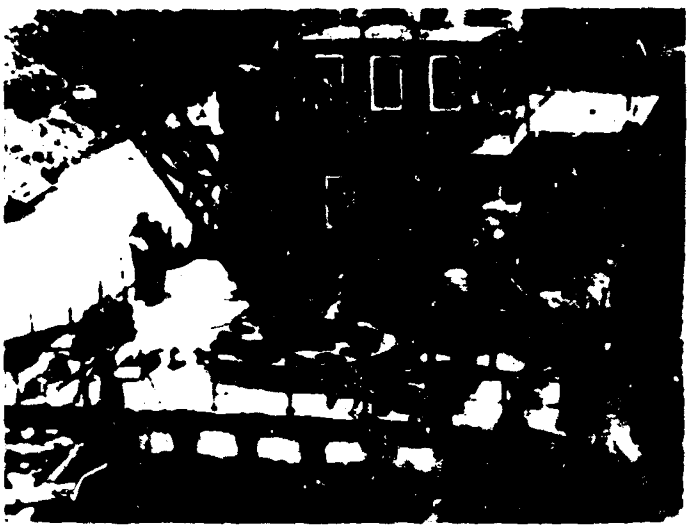


Fig.74. Pontoon-type Hoisting Crane during the Mooring Tests

Before the application of the paint, the concrete surface was cleaned of dirt and incrustations of corrosion with wire brushes; then it was blasted clean with compressed air.

The underwater part and the zone of the variable level were given three coats. In the area of the level of variable waterline and above it to the lower deck beam, on top of four layers of EKS-5

paint, we applied one layer of Kuzbass-lacquer. The paint was applied to the hull with a compressed-air painting device, while the Kuzbass-lacquer was applied with brushes.

After the second launching of the ship and the completion of all the finalizing operations, we conducted the mooring and cruising tests of the reinforced concrete pontoon-type crane (Fig.74). In this connection, we conducted observations of the design elements of the hull during the operation of the crane equipment, of the ship devices and systems, of the main engine and of the diesel generator. The operation of the devices, systems, diesel generator and also of the main engine during the mooring and cruising tests did not cause any disruptions in the hull designs.

After two years of operation of the pontoon-type hoisting /127 crane, the ship hull was in good condition. The coatings of paint applied to the hull were undamaged. No traces of corrosion through these coatings could be observed.

An example of the prefabricated-monolithic method of building the reinforced concrete ships is the barge with a lifting capacity of 1,000 tons, built in Czechoslovakia. The design of the barge hull has been discussed in Chapter II.

The basic technology of building the barge hull consisted in the following steps.

The floors, stringers, the transverse diaphragms of the space between the sides, and the ends of the barge were prepared on a timely basis and were delivered to the slipway in the form of finished flat and three-dimensional sections.

The bottom, the flooring of the second bottom, the outer and

inner sides were made directly on the slipway by the monolithic method. After the reinforcement of the bottom and the outer sides had been finished, on them we mounted the prepared sections of floors, stringers and frames, the reinforced outlets of which were fitted to the reinforced framework of the bottom and the side and were welded. We then concreted the entire external sheathing of the ships. In this operation, the sides were made monolithic through use of the formless method. In a similar way, on the floors, stringers and frames, we installed the reinforcement for the decking of the second bottom and the inner sides, and conducted their concreting.

The examples reviewed confirm the possibility of building the reinforced concrete ships by the prefabricated and prefabricated-monolithic methods at any enterprises engaged with the construction of ships from conventional ferroconcrete. In this connection, for the conduct of many technological operations (other than the specific ones referred to in Section 17), for the preparation of the prefabricated units and the shaping of the hull on the slipway, use can be made of the same equipment and gear as for the construction of the ships from the standard ferroconcrete.

/128

Chapter IV. ESTIMATING THE STRENGTH OF SHIPS DESIGNS OF REINFORCED CONCRETE HULLS

Sect. 20. Description of the Methods of Estimating the Strength of Reinforced Concrete Designs

The diversity of the thin-walled designs made of reinforced cement-sandy concrete is established by the large number of different systems for reinforcement and the forms of the reinforcing which is utilized. We find most often the thin-walled designs with combined reinforcement. In these designs, the cement-sandy type of concrete is reinforced with 2-4 layers of wire meshes and rod-type framework,moreover the stiffeners strengthening the design are reinforced only with rod-type supports. Also typical are the designs reinforced only with wire meshes uniformly through the section of the element.

For the characteristics of the reinforcement systems, we use the indexes: the relative content of steel arranged in the direction of interest to us (i.e. the percentage of reinforcement, μ), the type (rod and mesh reinforcement) and the arrangement of the reinforcement (concentrated and dispersed) specific surface of the mesh reinforcement, K_n.

In the use of one form of meshes, the value K_n is determined with the formula

$$K_n \simeq 5,65 \frac{d}{a} \cdot \frac{n}{h}, \qquad (4)$$

where d = the wire diameter of the meshes, cm;

a = the size of screen meshes, cm;

n = the number of screen layers in a section; and

h = the height of section, cm.

In this connection, between the reinforcement factor of the meshes (in percentages) and the specific reinforcement surface K_π, the relationship exists

$$\mu_c = 1.25 K_\pi d, \qquad (5)$$

where d = the wire diameter of screens, mm. /129

The diversity of the systems for reinforcing the thin-walled designs made of cement-sandy concrete and the considerable influence of the number of wire meshes on the nature of the appearance and the opening of the cracks in the concrete is explained by the fact that to the accomplishment of the estimations of the strength of reinforced concrete designs, two different approaches exist.

Thus, in the practice of onshore and hydrotechnical construction, the designs made of reinforced concrete with $K_\pi > 2$ cm^2/cm^3, we recommend the calculation by the methods of structural mechanics of the elastic systems, for the use of the elastic-strength characteristics obtained experimentally.

At a smaller value of the specific surface of the reinforcement (in spite of the fact that the limit $K_\pi = 2$ cm^2/cm^3 is very tentative, since the variation in the properties of the material with the change in K_π is accomplished gradually), the strength of the design made of cement-sandy concrete, reinforced with steel wire meshes is recommended to be calculated in the same way as the strength of the designs made of standard ferroconcrete, i.e. based on the three critical states; the supporting capability (strength,

stability, resistance to wear); based on the stresses; based on the formation and opening of cracks. The problem of the calculation is the provision for the given design of assurances against the development in it of any given critical state during the period of operation.

The estimation based on the bearing capacity for strength with consideration in the necessary cases of stability is conducted for all the designs; for the resistance to wear, for the designs occurring under the effect of a multiply recurring or pulsing load. The verification of the strength of the sections of the indicated designs is conducted similarly to the checking of the strength of the ferroconcrete designs: the operation of the concrete under elongation in a critical state is not taken into account. For the bending, eccentrically compressed or extended sections in a critical state, it is recommended to adopt the rectangular stress-strain diagram both for the entire extended reinforcement as well as for the entire compressed concrete (Refer to Sec. 22).

The calculation based on the stresses is conducted in those cases when in the design, considerable saggings can develop, obstructing the normal operation of the equipment or the floating facility as a whole.

The estimation of the formation and width of the cracks' opening is considered necessary when their appearance can put the equipment out of operation, or abruptly deteriorate its operational qualities.

In the reinforced concrete designs which we are considering, with a small number of meshes, the width of the cracks' opening is

checked according to the tentative marginal stress in the concrete for elongation, which is calculated for the extended, eccentrically loaded and bent elements based on the formulas for the resistance of materials. In this connection, the calculated area, the /130 inertial moment and the resistance moment are determined in dependence on the dimension and form of the concrete section of the element being checked, without reduction to homogeneity of the materials, while the permitted width of opening of the cracks at constant value of the provisional critical stress in the concrete is imposed in dependence on the quantity and number of the reinforcing screens.

For the domestic (Soviet) shipbuilding, the reinforced concrete is a new design material. The planning and construction of ships from reinforced concrete required the development of methods for calculating the strength of the ship designs.

In justifying the method for estimating the ship designs made of reinforced concrete, we took into consideration the following basic features of the material and the designs made from it:

1. The reinforced concrete designs are thin-walled. For practical purposes, their thickness ranges from 10-15 to 35-50 mm.

2. The reinforced screens of wire with a diameter of 0.5 - 1.2 mm are arranged uniformly over the entire height of the design's cross section. The thickness of the protective layer is slight. Based on the conditions of the watertightness of the sheathing and the protection of the mesh reinforcement from corrosion, the width of the cracks' opening is limited to the value of 0.05 mm.

3. In spite of the fact that reinforced concrete is a variant

of ferroconcrete, the process of its deformation differs significantly from that of standard ferroconcrete, specifically: by its resistance to cracking; by the greater stiffness prior to and after the formation of cracks; by the smoothness of the curve of saggings; by the rectilinearity (in the presence of breaking) of the stress strain diagram during elongation, bending, (separately for the extended and compressed zones) and the pure shear in respect to the stresses (refer to Figs. 6,7,14).

The rules developed for conducting the calculations of the strength of ship designs fror reinforced concrete establish a unified approach to the estimations of the strength of all the designs made cf cement-sandy concrete, reinforced by wire steel screens with a specific surface ranging from 0.5 to 3.0 cm^2/cm^3, and also by wire screens and of rod-type reinforcement together. The effect of the number and extent of dispersion of distribution of the screens is taken into account by the differentiated assignment of the standard resistances to elongation (axial and during bending) depending on the specific surface of the reinforcement.

The stresses originating in the reinforced concrete designs of a ship hull during the effect of a design load on it, are established according to the general rules of structural mechanics /131 under the assumption that the hull material is isotropic and under the effect of design loads, functions as a resiliant material. In this connection, the designs operating under flexure should be calculated with consideration of the difference in the values of the elastic characteristics for the compressed and extended zones of the section. In a number of cases, it is sufficient to know

only the ratio of the elasticity modulus to compression during bending, the elasticity modulus for elongation during bending, $E_{c.и}/E_{p.и}$. This ratio, in the range of stresses prior to the truncation of the diagram, can be assumed to equal unity, while in the range of stresses after the truncation of the diagram, the value of this ratio,other conditions being equal, depends on the value of the loads and the duration of their effect.

In the case of a brief relaxation of the operational load, it can equal 1.5 - 2.0, while in the case of a prolonged unloading, it can be 3.0.

The reinforced concrete designs, the extended sectional zone of which is additionally reinforced by the rod-type reinforcement, are also calculated by the methods of the resistance of materials and the theory of elasticity, but with consideration of the reduced section of the element. The latter is determined based on the ratio of the standard resistance of reinforced concrete and the calculated resistance of the rod-type reinforcement. The values of these resistances are limited separately for the extended and bent elements, proceeding from the conditions of the combined functioning of the rod reinforcement and of the reinforced concrete, and of the value of average deformations of reinforced concrete, with allowance for the cracks developing in them under the stresses assumed to be normal.

In the estimation of the elements functioning only under flexure, in which a part of the wire meshes in the central third of the height of the section is replaced by rod-type reinforcement, we decide to proceed from the actual geometric dimensions of the

section, and not from the reduced section, and to assume the standard characteristics of the material, considering the wire meshes to be arranged uniformly over the entire height of the section in such a quantity as in the extreme thirds of the section's height.

The combined designs, comprising a combination of beams made of standard concrete and plates of slight thickness reinforced by wire steel meshes should be calculated on the basis of the methods of disruptive loads, with allowance for the following feature: in the case of the location of the plate in the extended zone of the section, the disruptive force should be determined, proceeding from the reduced area of the plate, functioning together with the beam, and the values of the standard resistance of the reinforced concrete of the plate. The functioning of the beam concrete to elongation is not considered, while the extended reinforcement of the beam is introduced into the calculation with the stress, equalling the design resistance of the rod-type reinforcement in the reinforced concrete elements. /132

In addition to verifying the strength in respect to stresses, according to the established practice of conducting strength calculations of the ship designs made of reinforced concrete, we control the verification of the maximum stresses and also the stability of the design as a whole and of its individual units. In this connection, the compressed reinforced units are checked for stability only in the case that their flexibility $l_o/r > 50$, where l_o = the design length of the construction element; r = the least radius of the inertia of the cross section of the element.

The maximum value for the sagging of the reinforced concrete units (elements) from the operational load is determined based on the formulas of the resistance of materials, proceeding from the actual geometric dimensions of the design, and with the use of the reduced elasticity modulus of reinforced concrete in respect to saggings. The numerical values of the reduced elasticity moduli for sagging on the basis of experimental research (with the use of the grades 300 and 400 concretes) should be assumed equal under the brief action of the load: 200,000 kg/cm^2 for the reinforced concrete elements reinforced only with wire meshes, and 250,000 kg/cm^2 for the elements having combined reinforcement (the wire meshes and the rod reinforcement). Determining the value of the saggings at prolonged effect of the load, the specified values of the elasticity moduli for sagging should be reduced respectively by two and three times.

In the case of loads considerably exceeding the operating ones, for determining the saggings, it is recommended to use the reduced stiffness of the element, taking into account the difference in the elasticity moduli of the extended and compressed zones of the section, and the inertial moments of these zones.

Until the time when the procedures in the rules being applied in the planning of reinforced concrete ships will be verified by the prolonged practice of operating the ships, it appears feasible to combine the two methods of estimating the strength; i.e., based on the stresses, by the methods of structural mechanics (tentatively considering the reinforced concrete as a homogeneous isotropic material, and assuming its elastic-strength characteristics based

on the data from the tests, with allowance for the occurring crack formation) and based on the method of disruptive loads.

Although this will increase somewhat the scale of the calculations, it will permit us to obtain fairly reliable and economical designs from reinforced concrete, especially from the viewpoint of expenditure of steel.

The existing rules for the fulfillment of the strength calculations of the reinforced concrete designs of vessels do not touch on a number of questions not verified by the tests and practice. Taking this circumstance into account, let us introduce certain solutions, based on the theory of ferroconcrete and structural mechanics. /133

Sect. 21. Design of Beams, the Material of Which During Elongation and Contraction Follows the Hook Law, but During Elongation the Elasticity Modulus Does not Equal the Elasticity Modulus During Contraction

Distribution of Standard Stresses. The experiments indicate that in the iron-stone and concrete beams, the cross sections which are flat prior to bending will also remain flat after bending. In such a case, the elongations and contractions of the longitudinal fibers of the bending beam proved to be proportional to the distance from the neutral layer. Since in this connection, in the extended and contracted zones of the sections, the material followed the Hook law, although the values of the moduli in these zones are different, to the linear distribution of the stresses, there also corresponds, within the limits of each zone, a unique linear law of stresses' distribution.

The variation in the standard stresses along the beam's height is shown in Fig.75 by the line AOB. Assume that $E_{p\,м}$ and $E_{c.м}$ = the values of the elasticity moduli respectively for the extended and compressed sectional zones of the beam, the expressions for the maximum elongating and maximum compressing stresses will acquire the form:

$$\sigma_p = \frac{E_{p.м} h_p}{\rho}; \quad \sigma_c = \frac{E_{c.м} h_c}{\rho}, \tag{6}$$

where h_p, h_c = distances to the most remote fibers; and

ρ = the curvature radius

For a rectangular cross section of width b, the sum of all tensile and the sum of all contracting forces will be calculated with the formulas:

$$N_p = \frac{\sigma_p b h_p}{2}; \quad N_c = \frac{\sigma_c b h_c}{2}, \tag{7}$$

wherein

$$N_p = N_c \tag{8}$$

Having substituted in place of σ_p and σ_c, their expressions (6), we find

$$E_{p.м} h_p^2 = E_{c.м} h_c^2.$$

from which

$$\frac{h_p^2}{h_c^2} = \frac{E_{c.м}}{E_{p.м}}.$$

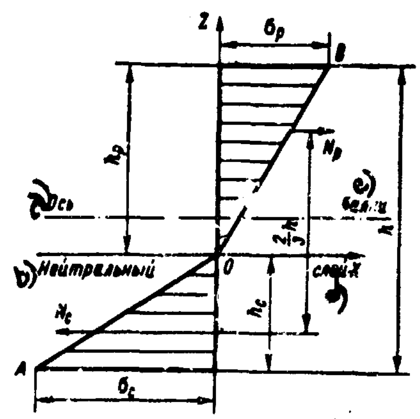
Having taken into consideration that $h_p + h_c = h$, we find

$$h_p = \frac{h\sqrt{E_{c.м}}}{\sqrt{E_{p.м}} + \sqrt{E_{c.м}}}; \quad h_c = \frac{h\sqrt{E_{p.м}}}{\sqrt{E_{p.м}} + \sqrt{E_{c.м}}} \tag{9}$$

/134

These equations determine the position of the neutral line when we know the ratio between the elasticity moduli for the extended and contracted zones of the beam's cross section.

Since all the forces distributed along the section reduce to a pair of sources, we determine their moment, multiplying the resultant of the tensile stresses times the arm of the couple 2/3 h (refer to Fig.75).



Key: a) axis; b) Neutral; c) beams; d) layer-X

Fig. 75. Distribution of Standard Stresses over the Height of the Beam, the Material of which During Elongation and Contraction follows the Hook Law, but $E_{p.и} \neq E_{c.и}$.

$$M = N\cdot\frac{2}{3}h = \frac{\sigma_p b h_p}{2}\cdot\frac{2}{3}h = \frac{\sigma_p b h_p h}{3} = \frac{\sigma_p b h^2}{3}\cdot\frac{\sqrt{E_{c.и}}}{\sqrt{E_{p.и}}+\sqrt{E_{c.и}}}, \quad (10)$$

from which

$$\sigma_p = \frac{3M}{bh^2}\left(1+\sqrt{\frac{E_{p.и}}{E_{c.и}}}\right). \quad (11)$$

Analogously, we derive

$$\sigma_c = \frac{3M}{bh^2}\left(1+\sqrt{\frac{E_{c.и}}{E_{p.и}}}\right). \quad (12)$$

Utilizing Eqs. (11) and (12), based on the prescribed **moment** of the **external** forces, we can find the value of the **maximum tensile** and compressing (contracting) forces.

Substituting in Eq. (10) in place of σ_p its expression (6), **ww** obtain the dependence between M and the curvature radius, ρ:

$$M=\frac{E_{\text{р. и}}h_{\text{р}}h^{2}b}{3\rho}\cdot\frac{\sqrt{E_{\text{с. и}}}}{\sqrt{E_{\text{р. и}}}+\sqrt{E_{\text{с. и}}}}=\frac{1}{\rho}\cdot\frac{bh^{3}}{12}\cdot\frac{4E_{\text{р. и}}E_{\text{с. и}}}{\left(\sqrt{E_{\text{р. и}}}+\sqrt{E_{\text{с. и}}}\right)^{2}}. \qquad (13)$$

The value $E_{\text{пр}}=\dfrac{4E_{\text{р. и}}E_{\text{с. и}}}{\left(\sqrt{E_{\text{р. и}}}+\sqrt{E_{\text{с. и}}}\right)^{2}}$ is said to be the reduced modulus of elasticity. /135

Equation (10) transforms to:

$$M=\frac{E_{\text{пр}}}{\rho}\cdot\frac{bh^{3}}{12},$$

from which

$$\frac{1}{\rho}=\frac{M}{E_{\text{пр}}I}. \qquad (14)$$

Using Eq.(14), based on the M-value and on the beam's **dimensions**, we can find the ρ-value.

The curvature $1/\rho$ is inversely proportional to the value of reduced stiffness, $E_{\text{пр}}I$.

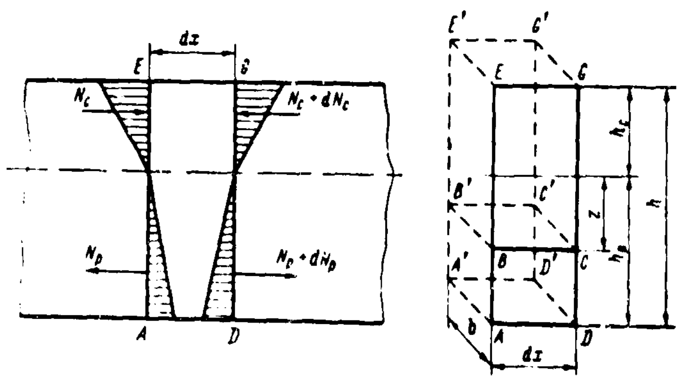


Fig. 76. Diagram Showing the Distribution of Standard Stresses in Two Infinitely Close Transverse Sections of a **Beam**.

The E_{np}-value depends on the ratio $E_{p.и}/E_c$.

$$\text{for } E_{p.и}/E_{c.и} = 0.5,\ E_{np} = 0.68E_{c.и};$$
$$\text{for } E_{p.и}/E_{c.и} = 0.1,\ E_{np} = 0.23E_{c.и}.$$

As is apparent, the absolute value of the corrected modulus E_{np} comprises a certain fraction of $E_{c.и}$, the value of which in turn changes at a variation in the grade of concrete.

The method discussed for investigating the distribution of the stresses can be readily extended to the sections formed from rectangles (for instance, the T- or double-T connections).

Distribution of Tangential Stresses. Having elucidated the question of the relative distribution of the standard stresses by height of the beam, we can also easily establish the nature of the distribution of the tangential stresses. Assume that AE and DG = = two infintitely close transverse sections (Fig.76), between which no external forces whatever are applied to the beam. /136

The tangential stress acting on area BCC'B',

$$dT = \tau b dx. \tag{15}$$

The standard (normal) stress in the elongated zone of section AE at level z_p from the neutral layer

$$\sigma_z = \frac{(\sigma_p)_1 z_p}{h_p}. \tag{16}$$

The normal tensile force N_p in section AE

$$(N_p)_1 = \int_{F_p} \sigma_z dF = \frac{(\sigma_p)_1}{h_p} \int_{F_p} z_p dF = \frac{(\sigma_p)_1}{h_p} S(z_p), \tag{17}$$

where S (z_p) = the static moment of the part of the area of extended zone of the section, the CG of which is determined by the ordinate z_p relative to the neutral line.

By analogy, the standard stress in the expanded zone of section DG

$$(N_p)_2 = \frac{(\sigma_p)_2}{h_p} S(z_p). \tag{18}$$

The difference in the standard stresses $(N_p)_2 - (N_p)_1$ is compensated during the projection onto the neutral axis with stress dT

$$dT = \tau b dx = \frac{(\sigma_p)_2 - (\sigma_p)_1}{h_p} S(z_p),$$

from which

$$\tau = \frac{(\sigma_p)_2 - (\sigma_p)_1}{h_p b dx} S(z_p). \tag{19}$$

Substituting into Eq.(19) instead of $(\sigma_p)_1$ and $(\sigma_p)_2$ their values according to Eq.(11), we find

$$\tau = \frac{3(M_2 - M_1) S(z_p)}{h_p b^2 dx h^2} \cdot \frac{\sqrt{E_{p.m}} + \sqrt{E_{c.m}}}{\sqrt{E_{c.m}}}.$$

Taking into account that $M_2 - M_1 = dM = Qdx$, and taking Eq.(9) into consideration, after transformations, we derive an expression for the distribution of the tangential stresses by height of the expanded zone of the beam's section:

$$\tau = \frac{QS(z_p)}{Ib} \cdot \frac{\left(\sqrt{E_{p.m}} + \sqrt{E_{c.m}}\right)^2}{4E_{c.m}}. \tag{20}$$

We derive the same by height of compressed (contracted) zone of the beam's section:

$$\tau = \frac{QS(z_c)}{Ib} \cdot \frac{\left(\sqrt{E_{p.m}} + \sqrt{E_{c.m}}\right)^2}{4E_{p.m}}. \tag{21}$$

The maximum tangential stresses correspond to the neutral /137 layer.

Substituting in (20) the values $I = bh^3/12$ and $S(z_p) = bh_p^2/2$ and taking Eq.(9) into account, we compute

$$\tau_{max} = 3/2 \cdot Q/bh. \tag{22}$$

Section 22. Estimation of Bearing Capacity of Reinforced Concrete Elements of Hull Designs Based on Formulas for Calculating the Ferroconcrete Designs

The critical state in respect to the bearing capacity develops at the time of the appearance of fluidity in the extended reinforcement or by the time of attainment of the critical values for the stresses in the compressed zone of the reinforced concrete element's section. In the case of a critical state, the functioning of the concrete in the elongated zone is not taken into account; the screen and the rod reinforcement is taken into account with its rated resistance. In view of the thin-walled state of the reinforced-concrete designs, the resistance of the concrete during axial and eccentric compression, and also in the compressed zone of the elements subjected to bending is determined by its prismatic strength. The values of the rated resistances of the cement-sandy concrete are assumed the same as for the heavy concrete of the pertinent brands, while the elasticity modulus is assumed with the coefficient 0.75.

The mechanical and elastico-plastic properties of the steel in the mesh networks formulated according to GOST 3826-47 or MTU-10-5-61 are not below the properties of brand St. 0 steel. The rated resistance for the screen according to GOST 3826-47 is assumed to equal 2100 kg/cm^2. The rated resistances of the rod-type reinforcement are adopted according to SNiP, Chapt. II-V-1, as for the ferroconcrete designs.

The bearing capacity of the reinforced concrete elements subjected to axial elongation and compression is calculated similarly to the calculation of the ferroconcrete sections. In this connection, in the calculation for elongation, we take into account the full section of the reinforcement of the rods and screens; in the calculation for

compression, the section of the fiber screens (owing to the danger of lamination of the material) is taken into account in an amount not exceeding μ_c = 1.0-1.5%.

A feature of the calculation of the bearing capacity of the ferroconcrete sections with the dispersed distributed reinforcement during bending and eccentric compression and expansion consists in the indeterminacy of the position of the center of the stretching forces in the section depending on the position of the neutral axis. Taking into account the fact that these stressed states are encountered most often in shipbuilding practice, let us examine the estimation of the carrying capacity of the bent, eccentrically compressed and elonga- /138
ted reinforced concrete elements based on the formulas for calculating the sections of the ferroconcrete designs.

In the derivation of the calculation formulas (with allowance for the indeterminacy mentioned in the position of the center of the elongating forces in the section with the dispersely distributed screen reinforcement), use was made of the method of converting the system of internal forces in a section for the critical state, which was first suggested by B.N. Samoylov*.

Calculation of Buckling Elements with a Section of Any Symmetrical Form Relative to the Plane of Flexure

The system of internal forces in a section for a critical state during bending and the arms of the internal couples are indicated in Fig. 77.

*B.N. Samoylov. Calculation of Elements of Reinforced Concrete Designs and Ferroconcrete Designs with Distributed Reinforcement. Publication of Kuybyshev Engineering-Construction Institute, 1964.

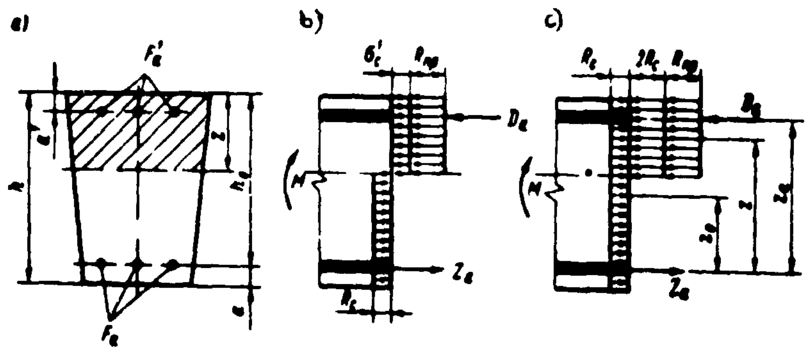


Fig. 77. Diagram of Internal Stresses in Section for Critical State During Flexure: a- geometric diagram of section; b- system of internal stresses for critical state; c- transformed system of internal stresses for critical state.

The equilibrium of projections of internal stresses in the section onto the elements axis is expressed by the equation:

$$Z_a + Z_c - D_c - D_б - D_a = 0.$$

The equilibrium of the moments internal stresses relative to the CG of the sectional area of the extended rod-type reinforcement is determined by the equation:

$$M + Z_c z_0 - D_a z_a - (D_c - D_б) z = 0.$$

In these equations:

Z_a - the critical stress in the rod reinforcement of the elongated zone of section, equalling:

$$Z_a = F_a R_a$$

where F_a - sectional area of rod-type reinforcement in the extended zone of section;

R_a - calculated resistance of rod-type reinforcement to stretching; and /139

Z_c - critical elongating force in all screens of the section equalling:

$$Z_c = F_c R_c = \mu_c F R_c = \mu a b h R_c,$$

where F_c - sectional area of longitudinal wires of all screens in section;

R_c - calculated resistance of screens' wire;

μ_c - coefficient of reinforcing with screens;

$\alpha = F/bh$ - characteristics of section;

b, h - parameters of width and height of section;

D_c - compressing force absorbed by the screens of the compressed zone of section, applied to the CG of the area of compressed zone and equalling;

$$D_c = F_c'(R_c + \sigma_c') = \mu_c F_б'(R_c + \sigma_c') = \mu_c \alpha_c bh_0 (R_c + \sigma_c'),$$

where F'_c - the sectional area of longitudinal screen wires, located in the compressed zone of the section;

$F_б'$ - the area of compressed zone of section;

$\alpha_c = F_б'/bh_0$ - characteristics of section;

σ_c' - stress in the screens' wire in the compressed zone of section, equalling R_c in the critical state, and

$D_б$ - critical compressing force in the concrete of the compressed zone of the section, applied to the CG of the comprssed zone and equalling:

$$D_б = F_б' R_{np} = \alpha_c bh_0 R_{np};$$

D_a - the critical compreesing force in the rod-type reinforcement of the section's compressed zone, equalling

$$D_a = F'_a R_{ac},$$

where F_a' - area of section of rod-type reinforcement in the compressed zone of section;

R_{ac} - calculated resistance of rod-type reinforcement to compression;

M - bending moment in section from external effects;

z_o - distance from CG of area of entire section to CG of sectional area of extended rod-type reinforcement, equalling z_o - $\gamma_o h$; /140

z_a - distance from CG of sectional area of rod-type reinforcement to CG of sectional area of extended rod-type reinforcement, equalling z_a - h_o - a', and

z - distance from CG of area of compressed zone of section to CG of sectional area of elongated rod-type reinforcement equalling:

$$z - \gamma h_o,$$

where γ_o - z_o/h; γ - z/h_o - coefficients of internal force couples in the section;

Introducing the notation $A_0 = \gamma\alpha_c = \frac{z}{h_0} \cdot \frac{F'_б}{bh_0} = \frac{S'_б}{bh_0^2}$, (where $S_б$' - the static moment of the area of the compressed zone of section relative to the CG of sectional area of the elongated rod-type reinforcement) and having substituted the above-listed expressions for the forces and arms into the equations of projections and moments of internal stresses in the section, after a series of transformations, we obtain seven calculation relationships.

The equation for determining the critical bearing capacity of the sections:

$$M = F'_a R_{ac} z_a + [2\mu_c R_c + R_{np}] A_0 bh_0^2 - \mu_c F R_c z_0. \quad (23)$$

The equation for finding the characteristic A_o

$$A_0 = \frac{M - \mu_c F R_c z_0 - F'_a R_{ac} z_a}{(2\mu_c R_c + R_{np}) bh_0^2}. \quad (24)$$

The equation connecting the characteristic A_o with α_c and γ, according to what is assumed above,

$$A_0 - \gamma\alpha_c \qquad (25)$$

The equation for determining the sectional area of the compressed rod-type reinforcement:

$$F_a' = \frac{1}{R_{ac}(h_0 - a')}\left[M - (2\mu_c R_c + R_{np}) A_0 b h_0^2 + \mu_c F R_c z_0\right]. \qquad (26)$$

The equation for determining the sectional area of the extended rod-type reinforcement:

$$F_a = \frac{1}{R_a}\left[(2\mu_c R_c + R_{np})\,\alpha_c b h_0 + F_a' R_{ac} - \mu_c F R_c\right]. \qquad (27)$$

The equation for determining the reinforcement factor with the screens

$$\mu_c = \frac{\alpha_c b h_0 R_{np} + F_a' R_{ac} - F_a R_a}{(F - 2\alpha_c b h_0) R_c}. \qquad (28)$$

The equation for determining the characteristic of the section

$$\alpha_c = \frac{\mu_c F R_c + F_a R_a - F_a' R_{ac}}{(R_{np} + 2\mu_c R_c) b h_0}. \qquad (29)$$

In particular cases when in a section there is only a single extended rod reinforcement and screens, in the calculation formulas (23) - (29) we should assume $F_a' - 0$; when there is only a compressed rod-type reinforcement and the screens--$F_a - 0$ and finally, in the case when th're are only screens, we should assume $F_a - F_a' - 0$.

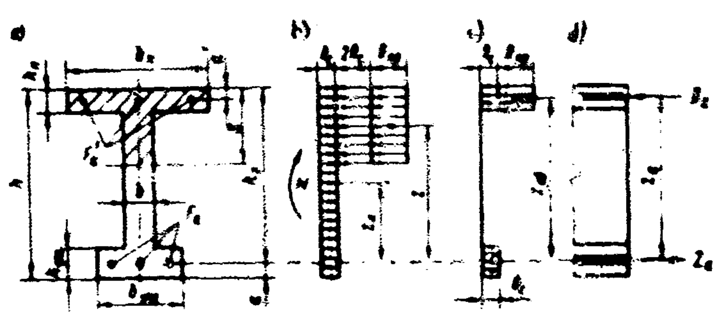


Fig. 78. Diagram of Internal Stresses in a Tee-Section for the Critical State During Bending; a - geometric diagram of section; b - system of inner stresses in rib; c - diagram of internal stresses in overhangs and widenings; and d - diagram of internal stresses in rod-type reinforcement.

Calculation of Bending Elements with Double-Tee, Tee and Rectangular Section. The diagram of forces and arms of internal force couples in a critical state during bending of a double-tee section is shown in Fig. 78.

The equilibrium of forces' projections onto the element's axis is expressed by the equation:

$$D_б^{св} + D_c^{св} + D_б^{р} + 2D_c^{р} + D_a - Z_c^{р} - Z_c^{уш} - Z_a = 0.$$

The equation of the moments of internal stresses relative to the CG of sectional area of the extended rod-type reinforcement

$$M - (D_б^{св} + D_c^{св})\, z_{св} - (D_б^{р} + 2D_c^{р})\, z - D_a z_a + Z_c^{р} z_0 - Z_c^{уш} z_{уш} = 0.$$

Substituting into these equations the values of the forces and arms, ensuing from the diagram in Fig. 78, specifically:

$$D_б^{св} = (b_п - b)\, h_п R_{пр}; \quad D_б^{р} = bxR_{пр} = \xi b h_0 R_{пр};$$
$$D_c^{св} = \mu_c^{св} (b_п - b)\, h_п R_c; \quad D_c^{р} = \mu_c^{р} bxR_c = \xi b h_0 \mu_c^{р} R_c;$$
$$D_a = F'_a R_{ac}; \qquad Z_c^{р} = \mu_c^{р} bhR_c;$$
$$Z_c^{уш} = \mu_c^{уш} (b_{уш} - b)\, h_{уш} R_c; \qquad Z_a = F_a R_a;$$
$$z = h_0 - 0{,}5x; \qquad z_{св} = h_0 - 0{,}5h_п;$$
$$z_0 = 0{,}5h - a; \qquad z_a = h_0 - a';$$
$$z_{уш} = 0{,}5h_{уш} - a,$$

and also introducing the notations

$$\eta = \frac{z}{h_0}, \quad \xi = \frac{x}{h_0}, \quad A_0 = \frac{xz}{h_0^2} = \xi\eta,$$

we arrive at the following calculation dependences.

The equation for the calculation of the critical bearing capacity of the section

$$M \le (R_{пр} + \mu_c^{св} R_c)(b_п - b)(h_0 - 0{,}5h_п)\, h_п +$$
$$+ (R_{пр} + 2\mu_c^{р} R_c) A_0 b h_0^2 + F'_a R_{ac} (h_0 - a') - \mu_c^{р} bhR_c (0{,}5h - a) -$$
$$- \mu_c^{уш} (b_{уш} - b)(0{,}5h_{уш} - a)\, h_{уш} R_c. \qquad (30)$$

/142

The equation for determining the characteristic A_0'

$$A_0' = \frac{1}{(R_{np} - 2\mu_c^p R_c)\, bh_0^2}\,[M - (R_{np} + \mu_c^{cn} R_c)(b_n - b)(h_0 - 0{,}5h_n)\, h_n - \\ - F_a' R_{ac}(h_0 - a') + \mu_c^p bhR_c(0{,}5h - a) + \mu_c^{yш}(b_{yш} - b) \times \\ \times (0{,}5h_{yш} - a)\, h_{yш} R_c]. \tag{31}$$

At the same time according to what was assumed above

$$A_0' = \xi\gamma = \xi\frac{z}{h_0} = \xi\frac{h_0 - 0{,}5x}{h_0} = \xi\left(1 - 0{,}5\,\frac{x}{h_0}\right) = \xi(1 - 0{,}5\xi). \tag{32}$$

Solving Eq. (32) for ξ, we find

$$\xi = 1 - \sqrt{1 - 2A_0'}. \tag{33}$$

The equations linking the characteristics A_0 and A_0',

$$A_0 = A_0' + A_0^{cn} = \xi(1 - 0{,}5\xi) + \frac{(b_n - b)(1 - 0{,}5h_n)\, h_n}{bh_0^2}; \tag{34}$$

$$A_0' = A_0 - \frac{(b_n - b)(1 - 0{,}5h_n)\, h_n}{bh_0^2}. \tag{35}$$

The sectional area of the compressed rod-type reinforcement:

$$F_a' = \frac{1}{(h_0 - a')\, R_{ac}}\,[M - (R_{np} + \mu_c^{cn} R_c)(b_n - b)(h_0 - 0{,}5h_n)\, h_n - \\ - (R_{np} - 2\mu_c^p R_c)\, A_0 bh_0^2 + \mu_c^p bhR_c(0{,}5h - a) + \\ + \mu_c^{yш}(b_{yш} - b)(0{,}5h_{yш} - a)\, h_{yш} R_c]. \tag{36}$$

The sectional area of extended rod-type reinforcement /143

$$F_a = \frac{1}{R_a}\,[(R_{np} + \mu_c^{cn} R_c)(b_n - b)\, h_n + (R_{np} - 2\mu_c^p R_c)\,\xi bh_0 + \\ + F_a' R_{ac} - \mu_c^p bhR_c - \mu_c^{yш}(b_{yш} - b)\, h_{yш} R_c]. \tag{37}$$

The coefficient of height of section's compressed zone

$$\xi = \frac{x}{h_0} = \frac{1}{(R_{np} - 2\mu_c^p R_c)\, bh_0}\,[F_a R_a + \mu_c^p bhR_c + \mu_c^{yш}(b_{yш} - b)\, h_{yш} R_c - \\ - F_a' R_{ac} - (R_{np} + \mu_c^{cn} R_c)(b_n - b)\, h_n]. \tag{38}$$

The expression for the characteristic α_c of section:

$$\alpha_c = \xi + \alpha_{cn} = \xi + \frac{(b_n - b)\, h_n}{bh_0}. \tag{39}$$

In this instance when the coefficients of reinforcement (by screens) of the flanges and the rib are identical

$$\mu_c^{ce} = \mu_c^{p} = \mu_c^{yw} = \mu_c,$$

Equation (37) is transformed

$$F_a = \frac{1}{R_a}\left[a_c(R_{np} + 2\mu_c R_c)\, bh_0 - \mu_c bhR_c + F_a' R_{ac}\right]. \tag{40}$$

Solving Eq. (40) for α_c, we obtain

$$\alpha_c = \frac{F_a R_a + \mu_c bhR_c - F_a' R_{ac}}{(R_{np} + 2\mu_c R_c)\, bh_0}. \tag{41}$$

The coefficient of reinforcement by screens at the prescribed sectional area of the rod-type reinforcement is found from Eq. (41), solving it for μ_c

$$\mu_c = \frac{a_c bh_0 R_{np} + F_a' R_{ac} - F_a R_a}{(F - 2\alpha_c bh_0)\, R_c}. \tag{42}$$

For the tee-sections with a flange in the elongated zone of the section, in Eqs. (30) - (42) we should assume $h_n = 0$; for the tee sections with a flange in the compressed zone- $h_{yw} = 0$; for the rectangular sections $h_n = h_{yw} = 0$.

Applicability Limits of the Calculation Formulas. The above-indicated calculation formulas were obtained under the assumption that the disruption of the bending moment begins from the elongated zone of the section, from the moment of advent of fluidity limit in the extended reinforcement. In those cases when the section in the extended zone is re-reinforced and the disruption of the element is limited by the advent of the critical stresses in the concrete and the reinforcement of the compressed zone of the element's section, at the stresses not reaching the critical ones in the extended zone, the calculation formulas cease being valid.

The condition of the original development of the critical stresses in the extended zone is expressed by the inequation known from the ferroconcrete theory: /144

$$S_{б}' \leqslant \xi S_0, \qquad (43)$$

where $S_{б}'$ - the static moment of area of compressed zone of section relative to the axis running through the CG of sectional area of the extended reinforcement;

S_o - the static moment of useful sectional area relative to the same axis; and

ξ - the coefficient depending on the type of concrete.

Th application of this condition to the sections with the dispersed-type distribution of the reinforcement is made difficult, since the position of the CG of the sectional area of the entire extended reinforcement depends in its turn on the position of the neutral axis, which complicates the calculation of the static moments $S_{б}'$ and S_o.

It is known that the maximum height of the compressed zone will take place in a rectangular section in which the screen reinforcement is lacking. In this case, condition (43) yields:

$$x \leqslant 0{,}55h_0.$$

In the case when in the section, the rod reinforcement is lacking and there are only screens, the maximum height of the compressed zone does not exceed $x = 0.50\ h_o$, since at a sufficiently close approximation of the compressed zone to this limit, the equilibrium of the internal stresses become impossible, while the section of the screen reinforcement tends toward infinity. This is indicative of the fact that at a large but finite reinforcement of the section of the reinforced concrete element by some screens, the

height of the compressed zone will be slight.

For the upper limit of the height of the section's compressed zone, determining the possibility of applicction of the reduced calculation formulas, one should adopt the least height of the compressed zone of the section, obtained from the following two conditions.

1. The height of the compressed zone of the section should not exceed half of the useful height of the section

$$x \leqslant 0{,}50h_0 \text{ or } \xi = \frac{x}{h_0} \leqslant 0{,}50. \tag{44}$$

2. The static moment of the area of the compressed zone of the section relative to the CG of the sectional area of the extended rod reinforcement should not exceed a certain part of the static moment of the useful sectional area relative to the same axis

$$S'_б \leqslant \xi S_0.$$

/145

The value of the coefficient ξ for brand 400 concrete is recommended to be assumed equal to 0.80, while for brands 500 and 600 concrete, the value should be assumed at 0.70 and 0.65, respectively.

For the tee-and double-tee sections, condition (43) is equivalent to the condition

$$A_0 \leqslant A_0^*, \tag{45}$$

where we determine the value A_0^* according to the formula

$$A_0^* = 0{,}5 + \left(\frac{b_п}{b} - 1\right)\left(1 - 0{,}5\frac{h_п}{h_0}\right)\frac{h_п}{h_0} + \left(\frac{b_{уш}}{b} - 1\right)\left(0{,}5\frac{h_{уш}}{h_0} - \frac{a}{h_0}\right)\frac{h_{уш}}{h_0}. \tag{46}$$

Application of Formulas to the Solution of Certain Problems on the Design of the Flexured Elements of Reinforced Concrete (in the example of an element of a double-tee section). a. Verifying the strength of the section. Given: the geometric characteristics of the section, the areas of the reinforcement sections, mechanical characteristics of materials, the bending moment, caused by the external forces.

1. Find the coefficient of height ξ of section zone according to Eq. (38).

2. Determine the characteristic A_c' with Eq. (32).

3. Find the bearing capacity of section M with Eq. (30).

4. The value of the moment based on Eq. (30) must be greater than the value of the moment from the external forces.

b. Determining the sectional area of reinforcement. Problem 1. The concrete of the compressed zone of the section was fully used, $A_o = (A_c)_{np}$.

Given are the geometric characteristics of the section, the reinforcement coefficient with screens, the mechanical characteristics of the materials and the bending moment in the section.

The finding of the sectional area of the extended and compressed rod-type reinforcement is conducted in the following sequence:

1. Assign as the critical value the characteristic A_o from condition (45).

2. Find the characteristic A_o' from Eq. (35).

3. Find the coefficient ξ from Eq. (33).

4. Find the characteristic of the section α_c from Eq.(39)

5. Determine the sectional area of the compressed rod reinforcement with Eq. (36).

6. Determine the sectional area of the extended rod reinforcement with Eq. (40).

Problem 2. The concrete of the compressed zone was not fully utilized, $A_0<(A_0)_{np}$.

The compressed rod reinforcement is prescribed, or is imposed by the design concepts. In addition, we know the geometric characteristics of the section, the coefficient of reinforcement /146 by screens, the mechanical characteristics of the materials and the bending moment in the section. The sectional area of the extended rod reinforcement is determined in the following order:

1. Determine the characteristic A_0' according to Eq. (31).

2. Find the coefficient ξ based on Eq. (33).

3. Find the characteristic of the section α_c from Eq. (39).

4. Determine the sectional area of extended rod reinforcement according to Eq. (40).

Problem 3. The concrete of the compressed zone of the section was not fully utilized, $A_0 < (A_0)_{np}$.

Known are the geometric characteristics of the section, the areas of the rod-type elongated and compressed reinforcement, the mechanical characteristics of the materials, and the bending moment in the section.

The sectional area of the screens' reinforcement or the coefficient of reinforcement by screens is found in the following order:

1. Determine the characteristic A_0' according to Eq. (31),

into which there is substituted the lease value μ_c, determined from the design concepts.

2. Find the coefficient ξ based on Eq. (33).

3. Determine the characteristic of section α_c based on Eq. (39).

4. Find the coefficient μ_c of reinforcement by screens based on Eq. (42).

In the case when the μ_c-value found in this manner differs markedly from the adopted for substitution into Eq. (31), it is re-substituted into this same formula, and we find the refined value for μ_c. Such a calculation by successive approximations, the number of which as a rule does not exceed 3, leads to the obtainment of a fairly close coincidence of the μ_c-values, calculated with Eq.(42) and adopted for the substitution into Eq. (31).

Calculation of Eccentrically Compressed and Eccentrically Extended Elements. 1. Case of Eccentric Compression. This is typified by the fact that the disruption occurs at the attainment of the critical values of stresses in the reinforcement of the extended zone of the section. Owing to the high plastic deformations associated with the onset of the phenomenon of fluidity in the elongated reinforcement, there occurs an intensive opening of the cracks, owing to which we have a decrease in the height of the compressed zone of the section; as a result, we have the development of critical stresses in the concrete and in the reinforcement of this zone of the section.

The system of internal forces for the critical state in case I of eccentric compression for an element with a section of any symmetrical form is shown in Fig. 79.

Writing an equation of the equilibrium of projections of internal forces in the section onto the element's axis, and the equation for the moments of these forces relative to the CG of the sectional area of the elongated rod-type reinforcement, after a number of /147. transformations, we will arrive at the calculation dependences for finding the bearing capacity of the section, and a determination of the sectional area of any of the reinforcements. These dependences are analogs of the Eqs. (23) - (29), typifying the critical state during flexure.

The equation for finding the critical bearing capacity of the section

$$Ne = (2\mu_c R_c + R_{np}) A_0 b h_0^2 + F_a' R_{ac} z_a - \mu_c F R_c z_0. \qquad (47)$$

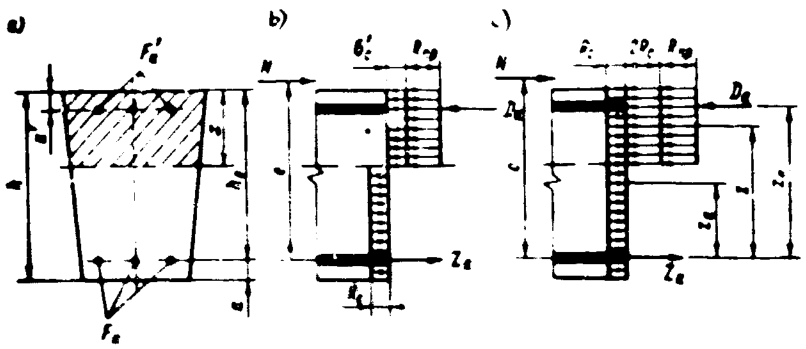


Fig. 79. Diagram of Internal Forces in a Section for the critical state in case I of eccentric compression: a- geometric system of section; b - system of internal stresses for critical state; and c - converted system of internal stresses for the critical state.

The equation for finding the characteristic A_0

$$A_0 = \frac{Ne + \mu_c F R_c z_0 - F_a' R_{ac} z_a}{(2\mu_c R_c + R_{np}) b h_0^2}. \qquad (48)$$

The equation linking the characteristic A_0 with α_c and γ,

$$A_0 = \alpha_c \gamma. \tag{49}$$

The equation for finding the sectional area of the compressed rod-type reinforcement

$$F'_a = \frac{1}{R_{ac}(h_0 - a')} \left[Ne - (2\mu_c R_c - R_{np}) A_0 b h_0^2 + \mu_c F R_c z_0 \right]. \tag{50}$$

The equation for finding the section area of the extended (elongated) rod-type reinforcement

$$F_a = \frac{1}{R_a} \left[(2\mu_c R_c - R_{np}) \alpha_c b h_0 - F'_a R_{ac} - \mu_c F R_c - N \right]. \tag{51}$$ /148

The equation for finding the coefficient of reinforcement by screens:

$$\mu_c = \frac{\alpha_c b h_0 R_{np} - F'_a R_{ac} - F_a R_a - N}{(F - 2\alpha_c b h_0) R_c}. \tag{52}$$

The equation for finding the characteristic of the section

$$\alpha_c = \frac{\mu_c F R_c - F_a R_a - F'_a R_{ac} - N}{(2\mu_c R_c - R_{np}) b h_0}. \tag{53}$$

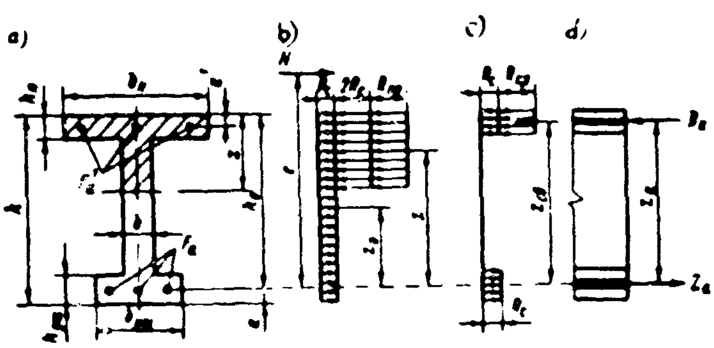


Fig. 80. Diagram of Internal Forces in a Double-Tee Section for the Critical State in case I of Eccentric Compression: a - geometric diagram of section; b - diagram of internal stresses in rib; c - diagram of internal stresses in the overhangs and widenings; and d - diagram of internal stresses in the rod-type reinforcement.

The diagram of the internal forces for the critical state in case I of eccentric compression for an element of a double-tee section is shown in Fig.80. The calculation dependences are derived from the equations of equilibrium of internal forces and moments.

The equation for finding the bearing capacity of the section

$$Ne = (R_{\text{пр}} + \mu_c^{\text{св}} R_c)(b_{\text{п}} - b)(h_0 - 0{,}5h_{\text{п}})h_{\text{п}} + (R_{\text{пр}} + 2\mu_c^{\text{р}} R_c) A_0' b h_0^2 + F_a' R_{\text{ас}} (h_0 - a') - \mu_c^{\text{р}} bhR_c (0{,}5h - a) - \mu_c^{\text{уш}} (b_{\text{уш}} - b)(0{,}5h_{\text{уш}} - a) h_{\text{уш}} R_c. \quad (54)$$

The equation for finding the characteristic A_o' and the coefficient ξ :

$$A_0' = \frac{1}{(R_{\text{пр}} + 2\mu_c^{\text{р}} R_c) b h_0^2} [Ne - (R_{\text{пр}} + \mu_c^{\text{св}} R_c)(b_{\text{п}} - b)(h_0 - 0{,}5h_{\text{п}})h_{\text{п}} - \quad (55)$$

$$- F_a' R_{\text{ас}} (h_0 - a') + \mu_c^{\text{р}} bhR_c (0{,}5h - a) + \mu_c^{\text{уш}} (b_{\text{уш}} - b)(0{,}5h_{\text{уш}} - a) h_{\text{уш}} R_c]; \quad (56)$$

$$A_0' = \xi(1 - 0{,}5\xi);$$

$$\xi = 1 - \sqrt{1 - 2A_0'}. \quad (57)$$

/149

The equations linking the characteristics A_o and A_o':

$$A_0 = A_0' + A_0^{\text{св}} = \xi(1 - 0{,}5\xi) + \frac{(b_{\text{п}} - b)(h_0 - 0{,}5h_{\text{п}})h_{\text{п}}}{bh_0^2}; \quad (58)$$

$$A_0' = A_0 - \frac{(b_{\text{п}} - b)(h_0 - 0{,}5h_{\text{п}})h_{\text{п}}}{bh_0^2}. \quad (59)$$

The sectional area of the compressed rod-type reinforcement

$$F_a' = \frac{1}{(h_0 - a')R_{\text{ас}}} [Ne - (R_{\text{пр}} + \mu_c^{\text{св}} R_c)(b_{\text{п}} - b)(h_0 - 0{,}5h_{\text{п}})h_{\text{п}} - (R_{\text{пр}} + 2\mu_c^{\text{р}} R_c) A_0' bh_0^2 + \mu_c^{\text{р}} bhR_c (0{,}5h - a) + \mu_c^{\text{уш}} (b_{\text{уш}} - b)(0{,}5h_{\text{уш}} - a) h_{\text{уш}} R_c]. \quad (60)$$

The sectional area of the extended rod-type reinforcement

$$F_a = \frac{1}{R_a}\left[(R_{np} + \mu_c^{ca} R_c)(b_n - b)h_n + (R_{np} + 2\mu_c^p R_c)\xi bh_0 + \right.$$
$$\left. + F_a' R_{ac} - N - \mu_c^p bhR_c - \mu_c^{yw}(b_{yw} - b)h_{yw}R_c\right]. \qquad (61)$$

The coefficient of the height of compressed zone of the section:

$$\xi = \frac{1}{(R_{np} + 2\mu_c^p R_c)bh_0}\left[F_a R_a + \mu_c^p bhR_c + \mu_c^{yw}(b_{yw} - b)h_{yw}R_c + \right.$$
$$\left. + N - F_a' R_{ac} - (R_{np} + \mu_c^{ca} R_c)(b_n - b)h_n\right]. \qquad (62)$$

The expression for the characteristic of the section α_c :

$$\alpha_c = \xi + \alpha_{ca} = \xi + \frac{(b_n - b)h_n}{bh_0}. \qquad (63)$$

In the case when the coefficients of reinforcement by screens of the edgings (?) and of the rib are identical:

$$\mu_c^{ca} = \mu_c^p = \mu_c^{yw} = \mu_c.$$

Equation (61) is transofrmed

$$F_a = \frac{1}{R_a}\left[\alpha_c(R_{np} + 2\mu_c R_c)bh_0 - \mu_c bhR_c + F_a' R_{ac} - N\right]. \qquad (64)$$

Having solved Eq. (64) for α_c, we obtain:

$$\alpha_c = \frac{F_a R_a + \mu_c bhR_c - F_a' R_{ac} + N}{(R_{np} + 2\mu_c R_c)bh_0}. \qquad (65)$$

The necessary coefficient of reinforcement by screens in case of a prescribed reinforcement of the rod-type mounting is found from a solution of Eq. (65) for μ_c

$$\mu_c = \frac{\alpha_c bh_0 R_{np} + F_a' R_{ac} - N - F_a R_a}{(F - 2\alpha_c bh_0) R_c}. \qquad (66)$$

In Eqs. (54) - (66), for the tee-joint sections with a /150 border in the compressed zone, we should assume $h_{yw} = 0$; for the tee-joint sections with a border in the extended zone, $h_n = 0$ and,

finally, for the rectangular sections - $h_n = h_{yw} = 0$.

Since the stressed state in case I of eccentric compression does not differ in its nature from the stressed state during bending, all the limitations of the height of the compressed zone of the section of the elements (which are being bent) are also extended to case I of eccentric compression.

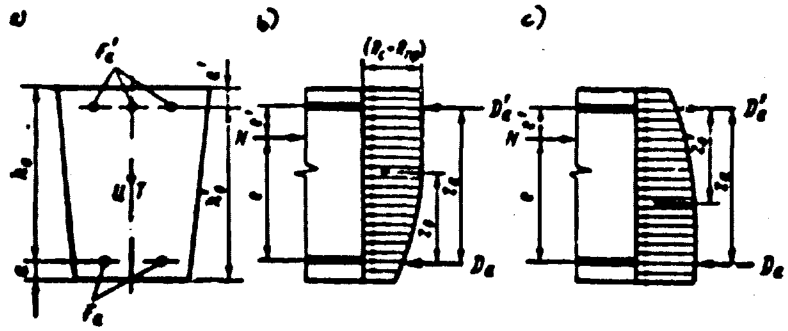


Fig. 81. Diagram of Internal Forces in a Section for the Critical State During Case II of Eccentric Compression: a - geometric diagram of section; b - case when the disruption starts from the edge closest to the force N; c - case when the breakdown begins from the edge more distant from the force N.

The design of the elements, extended eccentrically according to case I, can be achieved according to the formulas presented for case I of eccentric compression with the substitution into the design formulas of the value for the standard strength N with an opposite sign.

II. The case of eccentric compression is characterized by the fact that the disruption takes place upon the attainment of the critical stresses in the reinforcement and concrete of the most compressed sectional zone. The nature of the breakdown and the stressed state resemble the critical state for the centrally compressed elements.

In Fig. 81, we have shown a diagram of the internal forces for the critical state in case II of eccentric compression for an element with a section of any form, symmetrical relative to the flexure plane.

In the case of slight eccentricities, the breakdown can start both on the side closest to the force N, applied with the eccentricity e in relation to the CG of the sectional area as well as on the side more remote drom the force N, which will be determined both by the configuration of the section as well as by the areas of reinforcements F_a and F'_a. /151

The equation for the equilibrium of moments relative to the CG of the sectional area F_a of the rod-type reinforcement more distant from the standard force N, has the form

$$Ne = (D'_{\text{б}} + D'_c) z_{\text{б}} + D'_a z_a.$$

The equilibrium equation for the moments relative to the CG of the sectional area F'_a of the rod=type reinforcement located closer to the standard force N, has the form:

$$Ne' = (D_{\text{б}} + D_c) z'_{\text{б}} + D_a z'_a.$$

In these equations:

$$D'_{\text{б}} = FR_{\text{пр}}; \quad D_{\text{б}} = FR_{\text{пр}};$$
$$D'_c = \mu_c FR_c; \quad D_c = \mu_c FR_c;$$
$$D'_a = F'_a R_{ac}; \quad D_a = F_a R_a.$$

Having substituted the value of the internal forces and of the arms into the equilibruim equation of the moments, and having solved them relative to the force N, we will derive

$$N = \frac{1}{e} \left[(R_{\text{пр}} + \mu_c R_c) A'_0 b h_0^2 + F'_a R_{ac} (h_0 - a')\right]; \tag{67}$$

$$N = \frac{1}{e'} \left[(R_{\text{пр}} + \mu_c R_c) A''_0 b h'^2_0 + F_a R_a (h'_0 - a)\right]. \tag{68}$$

where

$$A_0^{\bullet}=\frac{F}{bh_0}\cdot\frac{z_0}{h_0};\quad A_0^{\bullet\bullet}=\frac{F}{bh_0'}\cdot\frac{z_0}{h_0'}.$$

From Eqs. (67) and (68), we can find the sectional areas of the rod-type fittings (reinforcement):

$$F_a' = \frac{Ne-(R_{np}+\mu_c R_c)\,A_0' b h_0^2}{R_{ac}(h_0-a')}; \qquad (69)$$

$$F_a = \frac{Ne'-(R_{np}+\mu_c R_c)\,A_0'' b h_0'^2}{R_a(h_0'-a)}, \qquad (70)$$

and also the necessary coefficient of reinforcement by screens, if the sectional area of the rod reinforcement is already known:

$$\mu_c = \frac{Ne-F_a' R_{ac}(h_0-a')}{A_0' b h_0^2 R_c}-\frac{R_{np}}{R_c}; \qquad (71)$$

$$\mu_c = \frac{Ne'-F_a R_a(h_0'-a)}{A_0'' b h_0'^2 R_c}-\frac{R_{np}}{R_c}. \qquad (72)$$

The application of the formulas to the solution of certain problems in the design of the eccentrically compressed elements made of reinforced concrete (in the example of an element of a double-tee section). Case I of eccentric compression.

a. Verification of sectional strength. /152

Given the geometric characteristics of the section, the sectional area of the reinforcement, the mechanical characteristics of materials, the external force in N and the eccentricity of its application, e.

1. Find the coefficient ξ from Eq. (62).

2. Determine the characteristic A'_0 from Eq. (56).

3. Find the bearing capacity of the section from Eq. (54) and compare it with the value of the moment, caused by the eccentrically

applied force, N.

b. Determination of sectional area of reinforcement. Problem 1. The area of the concrete of the compressed z one of the section is completely utilized, $A_o = (A_o)_{np}$.

Given the geometric characteristics of the section, the coefficient of reinforcing by screens, the mechanical characteristics of the materials, the force N and the eccentricity, e, of its application.

The determination of the sectional area of the extended and compressed rod-type reinforcement is conducted in the following order:

1. Assign the critical value of the characteristic A_o from the condition (45).

2. Find the value of the characteristic A'_o from Eq. (59).

3. Determine the sectional area of the compressed rod-type reinforcement from Eq. (60).

4. Find the coefficient ξ from Eq. (57).

5. Determine the section area of the extended rod-type reinforcement with Eq. (61).

Problem 2. The area of the concrete in the compressed zone is not fully utilized $A_o < (A_o)_{np}$.

The section of the compressed rod-type reinforcement is given, or is determined from the design concepts. We also know the geometric characteristics of the section, the coefficient of reinforcement by screens, the mechanical characteristics of the materials, the force N and the eccentricity, e, of its application.

We ascertain the sectional area of the extended rod-type reinforcement in the following sequence:

1. Find the characteristic A'_o from Eq. (55). /153

2. Find the coefficient ξ from Eq. (57).

3. Determine the sectional area of the extended rod-type reinforcement from Eq. (61).

Problem 3. The concrete in the compressed zone of the section is not fully utilized, $A_o < (A_o)_{\text{пр}}$.

We know the geometric characteristics of the section, the areas of the rod-type extended and compressed reinforcement, the mechanical characteristics of the materials, the force N and the eccentricity e of its application.

The sectional area of reinforcing the screen, or the coefficient of reinforcement by screens is found in the following order:

1. Determine the characteristic A'_o from Eq. (55), into which we substitute the least (from design concepts) value for μ_c.

2. Find the coefficient ξ from Eq. (57).

3. Determine the characteristic of section α_c from Eq. (63).

4. Find the coefficient μ_c of reinforcement by screens from Eq. (66).

If the value obtained for μ_c differs greatly from that adopted for substitution into Eq. (55), it is substituted into the same formula, and the calculation is repeated successively up to the obtainment of a sufficiently close coincidence of the μ_c-values, computed from Eq.(66) and being applied for substitution into Eq. (55).

Case II of eccentric compression. a. Verifying the strength of a section. Given the geometric characteristics of the section, the sectional area of the reinforcement, the mechanical characteristics

of materials, the external force N, and the eccentricities, e and e' of its application.

The verification of the bearing capacity of a section reduces to a comparison of the least of the values of force N, yielded by Eqs. 67 and 68, with the value N of the prescribed external force.

b. Determination of sectional area of reinforcement. Problem 1. We know the geometric characteristics of the section of the element, the coefficient of reinforcement by screens, the mechanical characteristics of the materials, the external force N, and the eccentricities of its application.

We determine the sectional areas of the compressed and extended rod-type reinforcement from Eqs. 69 and 70, respectively.

Problem 2. We know the geometric characteristics of the element's section, the sectional areas of the rod-type reinforcement, the mechanical characteristics of the materials, the external force N, and the eccentricity of its application.

We determine the required coefficient of reinforcement by screens based on one of the Eqs. 71 or 72, yielding the maximum value.

Section 23. Results of Tests for the Strength of Ship Designs Made of Reinforced Concrete

The effective utilization of reinforced concrete in the ship designs is conditioned by the reliability of our knowledge concerning its behavior in the designs at various values and methods of applying a load, and also by the reliability of the applied methods of calculating the strength of the reinforced-concrete designs. Proceeding from these concepts, for the purpose of obtaining experimental data concerning the operating features of the ship reinforced concrete designs,

we ran tests on the strength of several of them. In this connection, we incidentally investigated the influence of the systems of reinforcement, the operational capability of the connecting elements, and the effect of the manufacturing technology. /154

In connection with the fact that a large part of the designs in the makeup of the ship's hull functions for the absorption of the bending loads, the basic attention was devoted to the question of the investigation of the strength of the reinforced-concrete designs during this type of loads.

Bending of Beams. The tests for the bending of the reinforced-concrete plates, reinforced by ferroconcrete beams, from the viewpoint of working out a method for the estimation of the strength of such designs, the study of the influence of the reinforcement systems for reinforced-concrete plates were of primary interest.

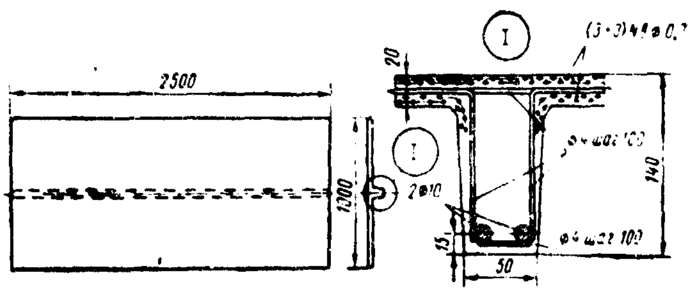


Fig. 82. Design of Ribbed Plates, Reinforced by One Stiffening Rib. Key: a) diameter 4, spacing 100.

We subjected to the tests, the designs of two types the plates reinforced by one rib (six designs of the type shown in Fig. 82) and the plates reinforced by two ribs for stiffening (three designs of the type shown in Fig.83).

For the production of the designs, we used sandy concrete, brand 400 on a base of sulfate-resistant Portland-cement, brand 500 with a composition (by weight) cement: sand: water = 1:2:0.36. The mobility of the concrete mixture corresponded to 5-6 cm of the settling of the cone used as a standard by the Stroy-TSNIL. The designs were reinforced by mesh-type steel screens (GOST 3826-47) and hot-rolled rod-type reinforcement (GOST 502-41).

The system for the reinforcement of the plates in the first type of design had three variants:

- reinforcement by ten mesh-type screens No. 8 with a diameter of 0.7, equally distributed through the thickness of the plates;

- the reinforcement of the intermediate welded mesh of rods with a diameter of 4 mm, located in the center of the plate's thickness, and with six webbed screens No. 8, diameter 0.7, with a fixed arrangement of three screens from each side of the intermediate rod-type screen; and

- the combined reinforcement similar to the previous case, with the placement of the screens one on the other without assembly tension, but with bonding into a bundle of binding wire. /155

The reinforcement of the plates in the second type of designs, and the reinforcements of the beams strengthening the plates are shown in Fig. 84. The concreting of the designs was conducted in the wood-metal forms with the rib downward, with utilization of type I-7 surface electric vibrators for packing the concrete. The anchoring of the ferroconcrete rib with the reinforced-concrete plate was accomplished by installing the lug of the clamps of the reinforcing frame of the rib between the half-packets of the webbed screens of the plate. All

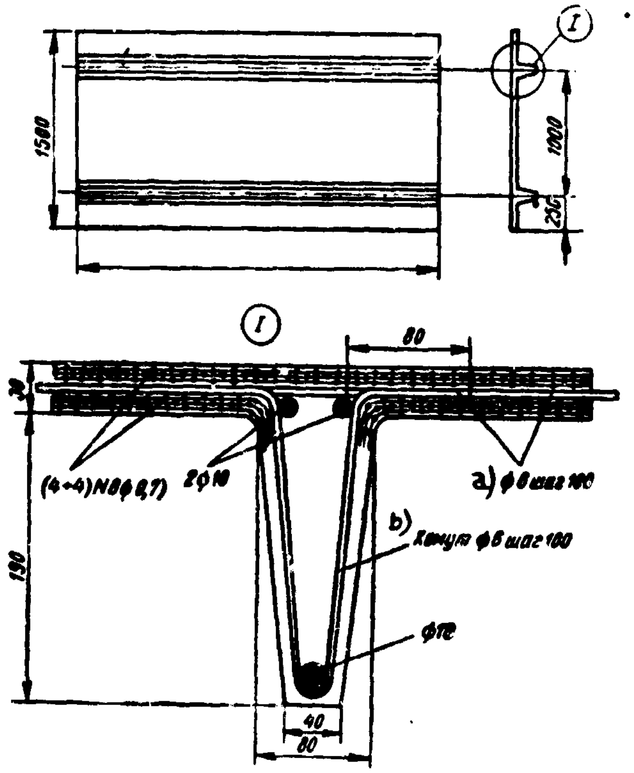


Fig. 83. Design of Ribbed Plates Reinforced by Two Stiffening Ribs. a) diameter 6, spacing 100; and b) clamp, diameter 6, spacing 100.

of the designs were tested in a condition when the reinforced-concrete plate was placed in the extended zone. The load was applied according to the system of pure bending, in stages of 1/10 - 1/15 of the breakdown load. At each stage of increasing the load, we measured the linear deformations and the sags, established the appearance of visible cracks, and we measured the width of their opening. The nature of the disruption of all the designs was identical /156
and corresponded to the following pattern.

The formation of the cracks started on the upper surface of the plates; then they penetrated the entire thickness of the plates and proceeded further into the extended (stretched) zone of the beams,

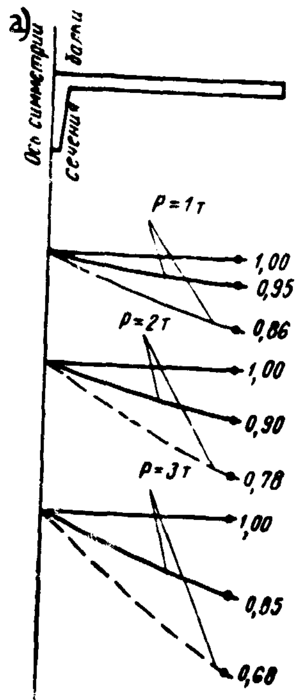


Fig. 84. Distribution of Stresses Across Width of a Sheathing Plate.

______ plate,reinforced only by webbed screens;

\- - - - plate, reinforced by webbed screens and by an intermediate welded screen of rods with a diameter of 5 mm. Key: a) symmetry axis of beam section.

reinforcing the plates. Under a further load of the designs, in the zones of the effect of the shearing forces, there appeared sloping cracks along the beams' walls. The width of opening of these cracks immediately reached 0.10 mm. The final rupture of the designs took place as a result of the breakdown of the compressed zone of the beams' concrete. By this time, the width of cracks' opening on the upper surface of the plate constituted 0.15-0.20 mm. We failed to observe ruptures in the webbed screens of the reinforced-concrete plates.

We succeeded in utilizing the results gained from measuring the linear stresses across the plates' width for the approximate determination of the value of the attached strip of the sheathing, participating jointly in the beam's functioning. We have indicated in Fig. 84 the distribution curves of the deformation across the width of the plates for different values of the loads based on the results obtained from testing the first type of designs. As is evident from the graph, the distribution of the stresses across the plates' width under all loads depends on the system of reinforcing the plates for those which are reinforced only by the webbed screens with $K_{\Pi} = 2.2$ cm^2/cm^3, we typically find a more uniform distribution of stresses by width than for the plates with the combined reinforcement having a specific reinforcement surface $K_{\Pi} = 1.7$ cm^2/cm^3.

Equating the area of the actual distribution curve of the stresses and the fictitious rectangular curve to the side, tantamount to a deformation in the plate directly over the beam, we determined the value of the attached strap. In the cases which are under consideration, the width of the attached strap comprised 92 cm ("pure" reinforced concrete) and 84 cm (combined system of reinforcing the plate), which exceeded by 35-25% the third part of the design span of the beams during the tests, and by 50-40% the twenty-fivefold actual thickness of the reinforced concrete plates. It is evident that the recommendations, generally adopted in the ferroconcrete shipbuilding, relative to the design width of a plate, functioning together with a beam, with application to reinforced concrete require some refinements. /157

For a judgment of the values of the loads at which the width of the cracks' o[illegible] in the reinforced concrete plates constituted

0.05 mm, and for a comparison of them with the calculated values, we have presented the data corresponding to the tests of the second type of designs:

Values of bending moments (ton-meters), corresponding to an opening of the cracks in the elongated zone by about 0.05 mm:
from the experiment 4.17
by calculation 4.75
Ratio of the values of the bending moment, obtained by the experimental and calculation method, % 88

The calculated value of the bending moment was determined on the basis of the method of disruptive loads, proceeding from the following assumptions: the uniform distribution of the forces through the thickness of the plate and the operation of the plate's material with the entire area of the section, to axial elongation. The adoption of these assumptions is substantiated by the nature of the deformation of the designs, expressed in the process of the tests.

The fairly close coincidence of the experimental and design values of the bending moment permitted us to recommend the calculation of designs, representing the combination of beams made of ferroconcrete and plates of slight thickness made of reinforced concrete, based on the principle of the method of breaking loads. In this context, in the case when the plate is located on the side of the extended zone of the section,the value of the breaking force and the value of the arm of the internal couple should be determined, proceeding from the reduced area of the plate and the standard resistance of the plate's material to axial elongation, adopting the curve of the stresses in the compressed zone of the beams according to a triangle (since at the moment of the

cracks' opening in the plate by ≈ 0.05 mm, as the experiments have shown, the equivalent compressing forces do not exceed 50% of its critical value). One should not take into account in the calculation the functioning of the concrete in the beams to elongation.

Bending of flat plates. The purposeful object of these tests was the obtainment of data on the strength of the monolithic reinforced concrete plates and the checking of the operating capability of the connection points of the reinforced concrete plates with each other and of the reinforced concrete plates with the ferroconcrete ones in one plane. The monolithic and jointed plates were subjected to the tests. All of the plates had the same dimensions in plan of 1500X1000 mm; the thickness of the plates and the systems of their reinforcement were different. /158

The connection of the reinforced concrete plates with each other and of the reinforced concrete plates with the ferroconcrete plates in one plane was conducted on a bypass without the welding of the reinforcement rods (the length of bypass was 75-80 mm) of the connecting plates, at simultaneous bypass into the zone of joining of the webbed screens. The design of the connecting reinforced concrete plates with the ferroconcrete ones corresponded to one of the variants of the design of plates of the 'tweendeck of the hulls on the ferroconcrete marine ships. The thickness of the ferroconcrete plates equalled 60 mm; the plates were reinforced with two screens from rods with a diameter of 6 mm.

The reinforced concrete plates were made from sandy concrete of the planned 500 brand. The consumption of materials per cubic meter

of cement-sandy concrete comprised:

Portland-cement, sulfate-resistant, C, kg	810
Sand, S, kg	1200
Water, W, liters	290
Water-cement ratio, W/C	0.36

The quality of the materials having been utilized for the preparation of the concrete complied with the requirements of the effective State Standards (GOST) for the building materials.

The data concerning the dimensions, system of reinforcement, strength of concrete, and number of tested plates are presented in Table 13.

The testing of all of the flat plates, jointed and monolithic, was conducted according to the system of pure bending. The load was applied in steps, equalling 0.20 of the calculated breaking load.

The duration of the delay between the individual stages comprised 5-10 minutes. During the delay, we measured the stresses, the bendings (sags) of the plates and the width of opening of the visible cracks. The breakdown of the plates took place from the side of the elongated zone in the action span of the maximum bending moment. The rupture of the extreme webbed screen in the elongated zone, as the final result of breakdown, was preceded by the formation and opening of cracks in the elongated surface of the indicated section of the plate.

The cracks' formation occurred at values of relative elongation stresses not exceeding the values $\varepsilon = 20-30 \cdot 10^{-5}$. Under a further load, the opening of the cracks which had formed was delayed owing to the increased adhesion of the reinforcement and the concrete, caused by the large surface of the webbed screens, and the increase in the stresses took place chiefly owing to the formation of the new

Table 13

Overall Dimensions, System of Reinforcement. Strength of Concrete and Number of Flat Plates Tested

Index of group of plates	Characteristics of designs	Planned dimensions of designs (mm)	System of the reinforcement	Number of designs	Strength of concrete to compression when testing designs kg/cm²
П-1	Monolithic plates of 'tweendeck	1500×1000×30	(4+4) № 8 ∅0,7, welded screen made of rods 6 mm in ∅, spacing 100mm	3	550
П-2	Monolithic plates of ceilings	1500×1000×20	(3+3) № 8 ∅0,7, welded screen of rods 4 mm in ∅ spacing 100 mm	3	525
П-3	" " "	1500×1000×12	5 № 8, ∅0,7	3	550
П-4	" " "	1500×1000×12	5 № 5, ∅0,5	3	550
ПС-1	Jointed plates of 'tween deck	1500×1000×30	The same as for plates П-1	3	550 (450)
ПС-2	Joining of reinforced concrete and of ferroconcrete plates	1530×1000×30/60	$\frac{\text{The same}}{\text{Two welded screens of rods 6 mm in ∅, spacing 100 mm}}$	2	$\frac{550\ (450)}{300}$

Remarks.

1. In the numerator, the data pertain to the reinforced concrete section; in the denominator, they refer to the ferroconcrete data.
2. In the parentheses we show the strength of concrete in the zone of joining.

cracks in such a way that in the action zone of the constant bending moment (M = const), the cracks up to the time of their opening by 0.05 mm were arranged with a spacing of 2-3 cm.

The strength of the reinforced plates in the flooring of the 'tweendecks and the plates of the hull ceilings of marine ships is sufficient (Table 14). /160

Table 14

Values of the Bending Moments Obtained by an Experimental and Calculation Method and also in Effect in Analogous Ship Designs Based on the Design Materials (by Groups of Plates).

Symbol for group of designs	Values of bending moments (ton-meters), corresponding to cracks' opening in elongated zone, by 0.05 mm		Value of effective moments in analogous ship designs according to calculations based on plans 800 & 803	Ratio of experimental values of bending moments to calculated, %	Ratio of values of bending moments obtained experimentally, to actual ones in the pertinent ship designs
	from experiment	based on calculation (design)			
П-1	0,116	0,142	0,05	82	2,3
П-2	0,783	0,740	—	106	—
П-3	0,256	0,335	—	76	—
П-4	0,433	0 460	—	94	—
ПС-1	0,116	0,145	0,03	80	3,8
ПС-2	0,116	0,142	—	82	—

Remark. The calculated values of the bending moments were determined according to the instructions of the "Tentative Rules for Conducting the Strength Calculations of Ship Designs of Reinforced Concrete."

The observed deviation in the experimental data toward the lower direction as compared with the calculation data can be ascribed to the increase in the thicknesses of the protective layer of the designs as compared with the planned thicknesses. The indicated

increase in the thicknesses of the protective layer is caused by the fact that all of the tested experimental designs had only plus deviations in respect to thickness, having reached 10-20% of the planned values of the plates' thicknesses, and at the same time, according to the adopted procedure for producing the designs, under which the webbed screens are previously combined into a bundle, the height of the reinforcement frame remained unchanged and was chosen for the planned design thickness.

The design of the connection in one plane of the reinforced concrete plates with one another, and of the reinforced concrete plates with the ferroconcrete ones, with the aid of a bypass without the welding of the reinforcing rods of the intermediate screens, and a bypass into the juncture zone of the webbed screens, provides in the juncture zone of the bending flat plates the crack resistance and strength which is equivalent to that in the flat (plane) monolithic plates. However, after the appearance of the first cracks, their /161
subsequent opening (considerably larger) and the breakdown of the jointed reinforced concrete plates during bending took place along the sections of the termination of the bypass zone of the reinforcing rods of the intermediate screen. This is evidently explained by the fact that during the loads causeing the formation of cracks in the elongated zone of the bending plates, there took place the disruption in the adhesion of the reinforcing rods with the concrete. The attempt to simplify the technology (abandonment of the welding of the projections of the intermediate screen) and to reduce the width of the joint to the minimal dimensions (reduction in the length of the bypass of the rods of the intermediate screen) was not reflected on the crack

resistance of the jointed plates as compared with the monolithic ones (refer to table 14). However, in the case of the bending of jointed plates, reinforced by beams, when the plate will be situated under conditions of elongation over the entire section, the indicated design of joints can not guarantee the crack resistance and strength, equivalent to that of the monolithic designs. For the reliable functioning of the connection of the sections in the designs of the indicated type, it is mandatory to weld the projections of the reinforcing rod in the junction zone.

Testing the reinforced concrete plates for compression. In connection with the fact that the reinforced concrete designs are the most thin-walled ferroconcrete designs, considerable interest for the practice was represented by the question of the effect of the flexibility of the reinforced concrete plates upon their bearing capacity during central compression.

We subjected to tests the plates with dimensions of 700 X 500 X 25 mm. The reinforcement of the plates consisted of (3+3) webbed screens No. 10, with a diameter of 1.0 mm and of an intermediate welded screen of rods with a diameter of 5 mm at a spacing of 100 mm. The plates were made from the cement-sandy concrete having the composition:

Portland-cement, brand 500, kg 750

Sand, kg 1350

Water, liters 260

Based on the data from the tests of the control samples up to the time of testing the plates, the strength of the concrete comprised 410 kg/cm^2.

The testing of the reinforced concrete plates for central compression was conducted under the hinged support of the short edges of the plates, which was attained by the utilization of special headers, with the consideration of which the calculated length of the plates comprised 750 mm. During the testing, the load distributed along the short edges of the plates was applied in stages. At each stage of loading, we measured the linear deformations of the plates with sensors (pickup devices) for resistance and we also used mechanical comparing devices.

It was established during the process of the tests that from the very beginning of loading the plates, their compression was accompanied by bending, in spite of the fact that they did not have, for practical purposes, an initial camber (round). Evidently this is ex- /162 plained by the dissimilar structure of the reinforced concrete through the thickness of the plates.

In the utilization of the combined system of reinforcement, the heterogeneity of the reinforced concrete structure is inevitable in the sense that the longitudinal rods of the intermediate welded screen are not located in the central plane of the plate. The heterogeneity. caused by the displacement of the welbed screens depends on the method adopted for producing the plates. The breakdown of the plates took place suddenly without the preliminary appearance of perceptible stresses or sags, and represented a brittle fracture, characterized by the pressing out of the concrete and by the crushing of the bared screens from the side of the most stressed edge, with the subsequent fracture of the screens on the opposing side.

The breakdown load, according to the average data from the testing of three plates, comprised 24.0 tons; according to the calculation without allowance for the flexibility, it was 41.5 tons; based on the calculation with allowance for felexibility equalling for the tested plates $l_0/b = 30$, the value of the critical load = 20.8 tons. The nearness of the values of the breakdown load based on the data from the tests, and of the critical load, with consideration of the plate's flexibility, confirms the need for taking into account the flexibility of the elements of the reinforced concrete designs operating under compression.

Section 24. Substantiation of the Standards of the Dangerous and Permissible Stresses for the Marine Reinforced Concrete Designs

In distinction from ordinary ferroconcrete, in the used of which in shipbuilding considerable experience has also been accumulated, reinforced concrete does not have any finished theory of adequately broad practice of application. Therefore, the justified standards for the reserve of strength for the marine reinforced concrete designs under the varying conditions of their loading have not yet been finally developed at the present time.

However, the planning and the construction of experimental ships made of reinforced concrete are impossible without specifying the standards for the strength reserve. In their development, we have taken into consideration the operating conditions of the designs in the makeup of a ship hull, and the features of reinforced concrete as a building material. The reinforced concrete, as a variant of ferro-

concrete, under elongation and compression, has differing values of the characteristics, corresponding to the beginning of the structural modifications of the material.

The process of the formation of cracks in reinforced concrete during elongation and bending depends appreciably on the type of reinforcement: of the dispersed (by webbed screens only) or combined (webbed screens and rods). /163

Even before the complete exhaustion of the strength, we find an abrupt increase in the lengthening of the cracks, with an intensive increase in their width of opening. The ratio of the stresses, corresponding to this state of the reinforced concrete, to the strength limit, established by the strength of the reinforcing frame, is all the higher, the higher the brand of concrete and the extent of dispersity of reinforcement, i.e., the higher the K_n.

The method for the calculation of the reinforced concrete marine designs, if we tentatively regard reinforced concrete as a homogeneous material, under the validity of the hypothesis of plane section for the elements with cracks in the extended zone is actually the calculation of the resistance to cracks.

The results of calculating the bearing capacity of the reinforced concrete elements, close to the experimental data, are obtained in the calculation based on the method of the breakdown loads.

In the reinforced concrete designs, by nature thin-walled, the protective layer is measured only in several milimeters,and the question of shielding the wire reinforcement having a large surface acquuires unique urgency, especially since the process of corrosion can cause the flaking and collapse of the protective layer of concrete, and the complete exposure of the reinforcement.

In this manner, based on the conditions of the corrosive strength of the reinforcement screens and the water impenetrability, the marine reinforced concrete designs are identical to the construction designs of the first and second category of crack resistance, in which during operation the visible cracks are not permitted. Therefore, as the dangerous stresses during elongation (axial and during flexure), for the ship designs made of reinforced concrete, we adopt the stresses corresponding to the width of cracks' opening of 0.01-0.05 mm at a thickness of the protective layer in the design comprising 2 mm. The dangerous stresses during compression and compression during bending are assumed equal to the prismatic strength of the cement-sandy concrete, i.e. without allowance for the screens located in the compressed zone of the section.

The values of the standard (dangerous) resistances of reinforced concrete are listed in the Appendix. For the permissible stresses, we assume a certain fraction of the standard resistances.

The strength standards for the ship designs made of reinforced concrete are specified to be such that in the transition to the strength limit of the material, the values of the reserve coefficients of strength in respect to breakdown are obtained as not below those established by the Rules for the Construction of Ferroconcrete Ships issued by the River Registry of the RSFSR and the Registry of the USSR. In the conversion of the values of the reserve coefficients to the standards of the permissible stresses, we took into consideration the relation between the stresses, corresponding to the moment of cracks' opening by 0.01-0.05 mm, and the strength limit (according to the experimental data for shipbuilding reinforced concrete, equalling 0.70-0.75).

Chapter V. STATIC TESTS OF THE STRENGTH OF REINFORCED CONCRETE HULL OF A FLOATING CRANE

Section 25. Purpose and Problems Involved in the Tests.

The pontoon driftwood-hoisting crane with a hull made of reinforced concrete (the characteristics of the hull design is given in Chapter 2) is the first and as yet the sole, not counting the small ships of sporting design, reinforced concrete ship in the practice of domestic (Soviet) shipbuilding. Prior to the lowering of the hull into the water, on the slipway of the dock, we conducted the static tests of the general strength of the hull, and the local strength of the section of the deck flooring. Taking into account that the introduction of reinforced concrete into the practice of ferroconcrete shipbuilding is inseparably linked with the requirement for refining the physical concepts concerning the functioning of the reinforced concrete designs of a ship hull and the consequent improvement of them, prior to the strength tests of the reinforced concrete hull, the problem was raised both of a purely research nature, as well as of a direct evaluation of the hull strength and of its individual units, the propriety and feasibility of the actual solutions, having found reflection in the process of planning the hull. Under the static tests for the strength, the following questions were solved:

1. The experimental checking of the overall hull strength during buckling and camber (hogging), of the local strength of the deck flooring section, and also the detection of the flaws in the hull-type designs.

2. A determination of the dependence $\sigma - \varepsilon$ in the basic longitudinal connections of the hull (sheathing of the deck and bottom)

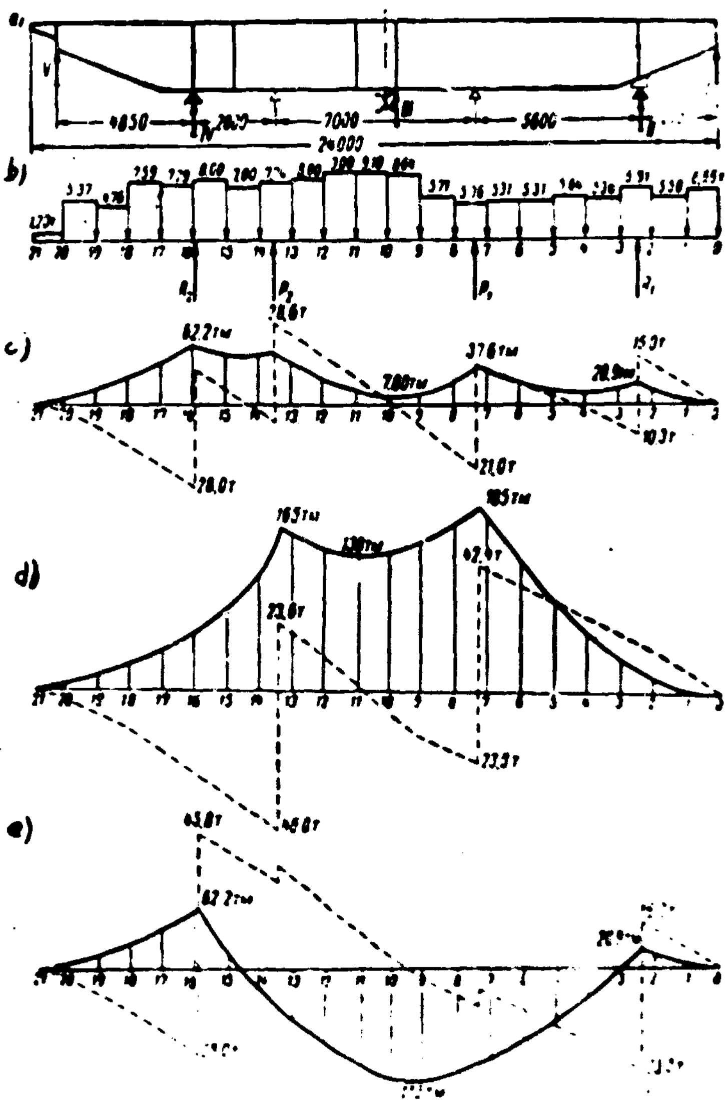


Fig. 85. Testing a hull for overall strength; a - system of arranging the supports along the length of hull and of sections for measuring the bending deflections.

Δ - mobile supports;
† - section of installing the sag measuring devices;
b - loads acting on the hull; c - curves of the bending moments and transverse forces in the hull prior to the beginning of the tests; d - curves of the bending moments and of the transverse forces during the camber of the hull (final stage); e - the same, during sagging of the hull (final stage).

during the general bending.

3. A determination of the elastic lines of the ship hull during its testing for buckling and camber (hogging).

Section 26. Procedure used for Conducting the Tests

The tests of the overall longitudinal strength were conducted by placing the hull onto four supports (Fig.85,a). The distance between the supports along the length of the hull was chosen from the condition of a uniform distribution of the hull weight on each support /165 in the position of the hull adopted during the tests for the zero position, and under the assumption that the hull weight corresponds /166 to that planned.

As the central supports, situtated in the sections along the 12th and 22nd actual frames (ribs), we utilized the building slip carts with hydraulic jacks having a lifting capacity of 60 tons. The terminal supports located in the sections along the 4th and 26th frames were stationary and represented the building slip carts and wooden cages.

The tests conducted on the overall longitudinal strength included the tests for camber and sagging. The bending moment was created owing to the actual weight of the hull by way of a graduated lifting or lowering of the central supports. At the time of the occurrence of hull only on the central supports, we determine the weight of the hull, which proved to equal 135 tons. The weight of the hull determined by weighing exceeded its planned value, corresponding to the saturation of the hull at the time of conducting the tests, by 29 tons. Taking into account that the excess weight of the hull was caused by the inaccuracy of the concreting, the amount of this overweight can be related

only to two items in the weight load: the hull and the superstructure (each of which is made of reinforced concrete).

Multiplying the ordinates of the indicated items of the weight load, corresponding to the planning data, times the coefficient found as the ratio of the weight of the hull and superstructure with allowance for the excess weight, to their weight based on the design data, we will derive new values of the ordinates for these items of the weight load.

In this manner, introducing the new ordinates for the indicatd items and having taken into consideration that all the other items of the weight load of the ship remained unchanged, we will obtain a distribution of the weights over the hull length during the conduct of the tests (Fig. 85 b).

The value of the load absorbed by each of the central supports was determined by multiplying the pressures in the cylinders times the area of the plunger in the lifting jacks of the building slip carts. Under the known values of the reactive forces of the central supports P_i, the reactions of the outermost supports for each stage of loading was determined from the equation of moments realative to each of the stationary supports, if we consider the hull as a beam, lying on them

$$\left(\sum M_{Q_i} + M_{R_1} + \sum M_{P_i}\right)_{R_2} = 0;$$
$$\left(\sum M_{Q_i} + M_{R_2} + \sum M_{P_i}\right)_{R_1} = 0.$$

where ΣM_{Q_i} = the bending moment from the actual weight of the hull (the hull weight within the limits of each theoretical frame spacing is distributed uniformly according to Fig. 96, b);

ΣM_{P_1} = the bending moment from the reactive forces of the central supports; /167

M_{R_1}, M_{R_2} = the bending moments from the unknown reactive force of one stationary support relative to the other.

The values of the reactions of the stationary R_1, R_2 and of the mobile P_1 supports by stages of load are given in Table 15. The nature of the change in the bending moments and in the transverse forces over the length of the hull is indicated in Fig. 85, c,d,e.

Table 15

Values of Reactions (Tons) of Supports by Degrees of Loading

Reactions in sections	Degrees of loading									
	Sagging					in position of hull prior to loading	Hogging			
	5	4	3	2	1		1	2	3	4
Along 4th sp. R_1	49,2	39,5	38	34,5	30,5	26,3	23,1	16,5	9	0
» 26 » R_2	73,8	69,5	57	52,5	46,5	40,7	33,9	26,5	16	0
» 12 » P_1	6	13	20	24	29	34	38	46	55	65,4
» 22 » P_2	6	13	20	24	29	34	38	46	55	69,6

The values of the bending moments, calculated from the position assumed during the tests for the zero position, in the sections of installing the measuring instruments by degrees of loading are indicated in Table 16.

Table 16

Values of the Bending Moments (Ton-meters) in the Hull Sections, Corresponding to the Places of Installing the Measuring Instruments During the Testing of a Hull for Overall Strength

Sections	Degree of loading									
	Sagging					In position of hull prior to loading	Hogging			
	5	4	3	2	1		1	2	3	4
A—A (between 11th & 12th spacings)	−120	−70.1	−61.1	−13.1	−22.1	0	16.1	51.3	69.5	[illegible]
Б—Б (between 14th & 15th spacings)	−120	−58.3	−59.5	−11.5	−20.2	0	18.0	[illegible]	[illegible]	[illegible]

NOT REPRODUCIBLE

/168

To the tests for local strength, we subjected the section of the deck flooring situated between the 4th and 7th frames, and also between the right side and the second longitudinal bulkhead. An evenly distributed load was created by spreading sand on the tested section of the deck, wherein the intensity of this load was increased in stages in the following sequence: 0.14; 0.30; 0.43 and 0.60 tons/m^2.

The bending moments for the under-deck beam in the span between the longitudinal bulkheads, established by calculating the rib frame, are presented in Table 17.

The bending moments for the plate of the deck (the latter is regarded as a nonsectional beam-strip with a width of 1.0 m) are presented in Table 18.

Table 17

Values of the Bending Moments in the Upper Branch of the Rib Frame Between the Longitudinal Bulkheads from Local Loading

Degree (stage) of loading	Intensity of running load on frame, ton-meters	bending moment, ton-meters		
		in support section	in section 300 mm from support section	along center of span
I	0,098	0,122	0,061	−0,084
II	0,210	0,294	0,160	−0,171
III	0,305	0,401	0,219	−0,233
IV	0,420	0,556	0,306	−0,317

Table 18

Values of Bending Moments in the Deck Plate Local Load

Degree of loading	Intensity of running load, ton-meters	Bending moment, ton-meters		
		in support section	in section of installing the tensometers	along center of the span
I	0,098	0,00712	0,00312	−0,00197
II	0,210	0,0152	0,00667	−0,00430
III	0,305	0,0221	0,00969	−0,00612
IV	0,420	0,0305	0,0134	−0,00856

The loading of the hull was done by stages during the tests. After each of them, a delay was made of 10-15 minutes, necessary for the stabilization of the stress-strain state of the hull. Under the /169 effect of the maximal bending moments, the hull was kept for one hour.

At each stage of the loading, during the testing of the overall and local hull strength, we measured the linear deformations of the hull connections (joints), we determined the elastic line of the hull during general bending, and we measured the pressure of liquid in the plungers of the building slip carts.

The measurement of the linear deformations during the general bending of the hull was conducted with comparators, which were installed on the bottom and deck in the section between the 11th and 12th frames during hogging and in the section between the 14th and 15th frames during sagging.

Under the arrangement of the devices, in addition to the value of the effective bending moment, we took into account also the remoteness of the section from the region of the abrupt changes in the form of a transverse section, caused by the presence of the superstructure and the notch of the hull.

During the tests of the strength of the deck to bending, by local loading, the linear deformations were measured by tensometers of the Gugenberger type which were mounted half-way along the length of the side of the support edge of the plate parallel to the short side. In the checking of the strength of the deck beams during bending by local load, the comparators measuring the linear stresses were installed in the central and in the outer sections of the beam span parallel to the beam's axis.

The measurement of the sagging deflections during the static tests of the hull was conducted in five different sections, coinciding with the arrangement of the transverse bulkheads, which excluded the possibility of the deformation of the hull sections under the load in

the process of testing. The depths of camber were measured by the sag measuring devices of the Maksimov system, fastened to the stationary designs, not connected with the hull, along each of the sides. The system of arranging the sag measuring devices along the length of the hull is indicated in Fig. 86, a. For determining the reactive forces transmitted to the hull by the mobile supports, the pressure in the hydraulic jacks of the building slip carts was measured carefully with calibrated manometers.

Section 27. Results Obtained from the Tests

In the inspection of the hull prior to the tests, we established that certain beams in the deck are non-coaxial (up to 40 mm) with the ribs of the sides and the longitudinal bulkheads. In addition, the beams of the framing had chipped-off places and nonconcreted areas. In the sections of the bottom and joints of the plates, there projected for 15-30 mm the coverings and excrescences of concrete. In a number of places, the protective concrete layer above the webbed screens was lacking. /170

In the testing of the hull for sagging, the maximal bending moment equalled 113 ton-meters, while in testing the hull for camber (hogging), it was 185 ton-meters (Fig. 86,e), which exceeded by 4.38 and 1.70 times respectively the calculated bending moments for the foot and crest of the wave.

The results obtained from measuring the linear deformations and saggins of the hull (Tables 19,20,21), and also the behavior of the hull in the process of the tests indicate that the design of the hull, even in the presence of the above-enumerated defects, caused by

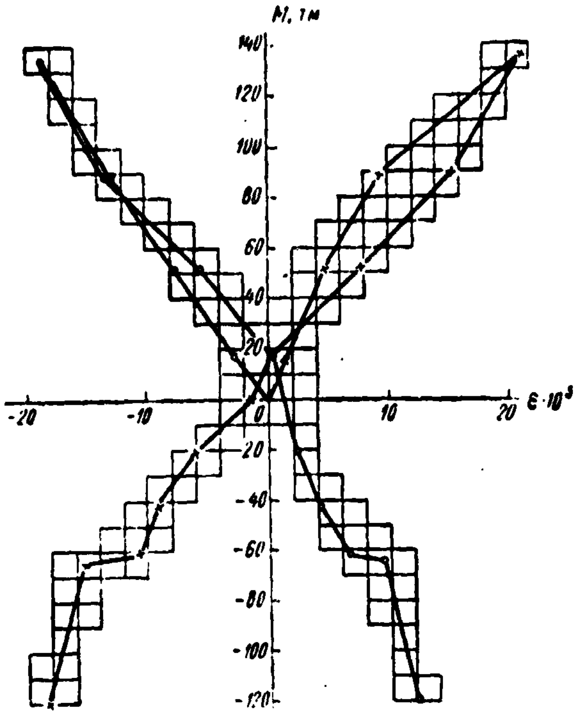


Fig. 86. Dependence of the load-deformations for the plates in the deck and bottom during the overall bending of the hull. X - comparator 21; O- comparators 16,17. Key: a) M, tons-meters.

the disruptions in the construction process, provided a monolithic state of the hull and the participation of all of its essential members in absorbing the loads acting on the hull. No cracks or any other visible damages to the hull in the process of the tests were recorded.

The relationship curve M - ε for the plates of the deck and the bottom during the overall bending is indicated in Fig. 86. In /172 Fig. 87, we have indicated the curves of the variation in the saggings of the hull in a position on the two central supports during camber and of the two outer supports during sagging under the effect

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Table 19

Relative Linear Stresses $\varepsilon \cdot 10^5$ of Plates in Deck and Bottom During Tests of Hull for Overall Strength

Nomenclature of Design	Bending moment, ton-meters													
	0	Hogging (Overbend)							0	Sagging				
		16,4	51,3	89,5	138	89,5	51,3	16,4		—22,1	—43,1	—61,2	—70,1	—120
Deck plate (comparator 21)	0,00	1,51	1,00	9,22	21,6	15,4	7,70	0,00	—1,51	—6,15	—9,22	—10,8	—15,4	—18,4
Flooring plate (average of readings from comparators 16 & 17).	0,00	—1,96	—7,71	—13,0	—18,8	—13,4	—5,10	0,41	1,36	2,03	1,05	6,16	9,28	12,1

Remark. For start of reading, we adopt the stressed state of hull (Fig. 84, c).

Table 20

Relative Linear Stresses $\varepsilon \cdot 10^5$ of Deck Plate and Sub-Deck Beam During Flexure by Local Loading

Extent of loading	Intensity of running load, ton-meters	Relative linear stresses $\varepsilon \cdot 10^5$					
		Deck plate				Sub-deck beam	
		Tensometer 4	Tensometer 5	Tensometer 6	Average of gp. of tensometers	Comparator 1	Comparator 2
I	0,098	—0,87	—0,94	—	—0,90	4,02	1,00
II	0,210	0,00	—	0,00	0,00	5,36	6,66
III	0,305	3,48	2,80	2,56	2,94	8,04	6,66
IV	0,430	8,70	6,55	4,70	6,65	10,7	10,00

of its own weight. The construction of the elastic lines was made from a position assumed to be the zero one.

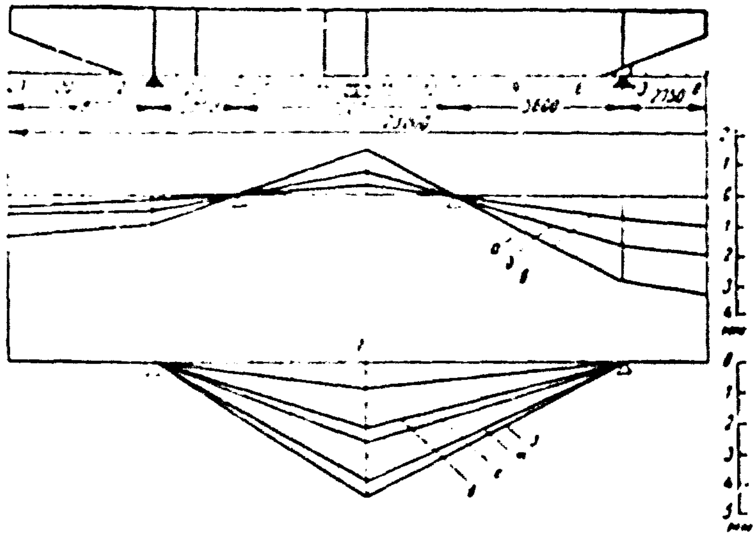

Fig. 87. Line of the hull saggings during general bending: a,b,c - 2,3, 4- degrees of camber; d,e,f,g,h - 1-5 degrees of saggings.

The calculation of the stresses in the hull members under the effect of the test loads was conducted with allowance for a certain thickening in the sheathing plates as compared with the planned thicknesses, owing to the inaccuracies in the concreting; specifically, as the design values of the sheathing thickness for the plates of the bottom, sides and deck, we adopted the value of 26.5 mm, while for the bulkhead plates, we assumed 21.0 mm. The variation in the

Table 21

Readings of the Sagging Meters (cm) During the Overall Bending of the Hull

Sections of location of sagging meters	Degrees of loading										
	Hogging					Sagging					
	0	1	2	3	4	0	1	2	3	4	5
Sp. 0	0.00	−0.05	0.18	−0.40	[illegible]	0.00	0.12	0.21	0.30	0.45	[illegible]
" 4	0.00	−0.04	−0.20	−0.33	[illegible]	0.00	0.12	0.23	[illegible]	0.42	[illegible]
" 16	0.00	−0.08	−0.31	−0.68	1.34	0.00	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
" 26	0.00	−0.09	−0.28	−0.50	1.19						
" 33	0.00	0.16	0.25	[illegible]	[illegible]	—					

Remark. The zero readings of the instruments correspond to the stressed state of the hull (Fig. 84, c).

linear stresses in the extreme fibers of the equivalent beam (of the deck and bottom plates) during the overall bending of the hull can be estimated with sufficient accuracy by straight lines (see Fig. 86). In this connection, the linear deformations in the compressed and elongated zones are almost equivalent, and do not exceed the absolute values $(10 - 20) \cdot 10^{-5}$. Based on what has been said, in a calculation of the stresses in the hull members, we can assume as identical or very close the values for the moduli of the extended (elongated) and compressed zones of the transverse section of the hull. The values of the stresses in the hull during overall bending, obtained as a result of calculation under the above-mentioned assumptions, are presented in Table 22. All the reinforced concrete elements were introduced into the calculation with there own reduced area, i.e. with allowance for the additional reinforcement of these elements of the reinforcing rods. The reduction factor of the rod (extended and compressed) reinforcement to the reinforced concrete $n = 10$.

Table 22

Values of Stresses in the Plates of the Deck and Bottom (at the Level of Installing the Comparators) from the Overall Bending of the Hull

Nomenclature of design	Bending moment, ton-meters								
	Hogging				Sagging				
	16.4	51.3	39,5	138	−22.1	−43.1	−61,2	−70,1	−120
Deck plate	1.88	5.88	10.3	15.8	−2,53	−4,94	−7,01	−8,03	−13.8
	1.80	5.66	9,83	15.2	−2,82	−5,50	−7.80	−8,94	−15,3
Bottom plate	−2,02	−6.33	−11,1	−17,0	2,73	5,32	7.55	8.65	14.8
	−2,09	−6.53	−11,4	−17.6	2.43	4,74	6.72	7.71	13,2

Remark. The figures in numerator correspond to calculation at identical values of elasticity moduli for the compressed & elongated zones of section $E_c./E_p.$ 1; numbers in denominator match the calculation at various values of elasticity moduli for compressed & elongated zone of section $E_{c.n}/E_{p.n} = 1.5$.

The very slight stresses from the overall bending of the hull are quite regular, if we take into account that the thickness of the sheathing for the small ships is specified on the basis of the conditions providing local strength, and not a general strength; moreover, in the planning of a hull for a pontoon crane, the sheathing thickness was specified not from a calculation of the stressed state, but from a design standpoint, for the purpose of increasing the resistance to impact.

The suggestion concerning the diversity in the elasticity moduli for the elongated and compressed zones of the section leads to a redistribution of stresses in the following manner: the elongation /174 stresses diminish, while the compression stresses increase as compared with the stresses determined under the assumption of the identical values of $E_{c.м}$ and $E_{p.м}$. At the ratio $E_{c.м}/E_{p.м} = 1.5$ adopted in our calculation, the indicated redistribution of stresses is quite insignificant.

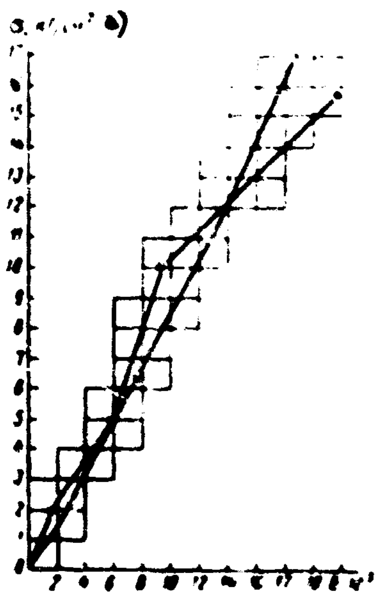

Fig.88. Stress-strain diagram for the plates in the deck and bottom during the overall bending of the hull. ● - deck plate; X - bottom plate. Key: a) kg/cm^2

The curve σ-ε, constructed on the basis of the data in Tables 22 and 19 is shown in Fig. 88. The values of the stresses in the sub-deck beam from the bending by the local load, calculated under similar assumptions are presented in Table 23. Table 24 shows the values of the stresses in a deck plate from the bending by a local load, determined from the formula $\sigma = M/W$.

The calculated values of the saggings of a ship hull during overall longitudinal bending were also determined with consideration for the increase in the dimensions of the cross section of the hull members, caused by the inaccuracy of the concreting. In this connection, we assumed that all the hull members were situated in the stage of the elastic functioning of the concrete, when cracks were lacking.

A comparison of the values of the saggings having taken place during the process of the tests in the final stages, with the calculated values of the saggings for these positions of the hull (obtained when proceeding, on the one hand, from the value of the reduced modulus for sagging $E_{\text{пр}} = 100{,}000$ kg/cm^2 and, on the other hand from the reduced stiffness of the cross section of the hull at $E_{\text{с.и.}}/E_{\text{р.и.}} = 1.50$) indicates their fairly close approximation and undoubtedly confirms the acceptability of the formulas from structural mechanics in a determination of the deformations (saggings) of the reinforced concrete designs.

An analysis of the procedure used and the results obtained from the tests of the overall strength of a pontoon crane hull made of reinforced concrete, and also of the local strength of the section of the deck flooring permitted us to make the following con-

clusions.

1. The overall longitudinal hulls strength, and also the local strength of the section of deck covering under the effect of loads developed during the tests were fully provided.

Table 23

Values of Stresses in the Sub-deck Beam from the Bending by a Local Load

Stages of oloading	Intensity of running load, ton-meters	Stress, kg/cm^2		
		in rack (on support 3)	in rib in span(in place of instal-ling comparator 1)	at support (in place of installing comparator 2)
I	0,098	8.55/7.95	16,8/25.8	−12,2/−13.9
II	0,210	20.7/19.2	34.2/52.5	−32,0/−36.5
III	0,305	28,2/26,2	46,6/71.6	−43,8/−50,0
IV	0,420	39,2/36,3	63,4/97.3	−61,2/−69.8

Remark. Numbers in numerator pertain to calculation at identical values of elasticity moduli for compressed & elongated zones of section $E_{c.н.}/E_{р.н.} = 1$; numbers in denominator refer to calculation at various values of elasticity moduli for compressed & elongated zones of section $E_{c.н.}/E_{р.н.} = 1.6$.

Table 24

Values of Stresses in a Deck Plate from Bending by a Local Load

Stages of loading	Intensity of running load, ton-meters	Stress, kg/cm^2		
		in support section	in section of installing the tensometers	along the center of the span
I	0,098	6,09	2.66	1,68
II	0,210	13,0	5,50	3,67
III	0,305	18,8	8,19	5,22
IV	0,420	26,1	11,4	7,31

2. The conformity of the results obtained from measuring the hull saggings during the overall bending with the calculated data according to the determination of the displacements of the hull in the process of the tests based on the formulas from structural mechanics during the application of the reduced elastic modulus in respect to the sagging $E_{пр}$ = 100,000 kg/cm^2 and of the reduced stiffness of the hull should be regarded as a known confirmation of the acceptability of the methods, recommended by the Provisional Rules for the calculated estimation of the deformations (saggings) of the reinforced concrete designs. /175

3. The results of the tests according to the determination of the relative linear deformations and stresses are of slight interest, which is explained by the very low quantity of data and small values of the stresses and strains, having taken place in the hull during the general bending.

APPENDIX

Provisional Rules for the Conduct of Calculations of the Strength of Ship Designs Made of Reinforced Concrete

I. GENERAL CONCEPTS

1. The Present Rules apply to the ship designs, made of cement-sandy concrete and reinforced steel screens or steel screens and reinforcing rod, in accordance with the instructions in Sections II and V.

2. We should consider as reinforced concrete designs the dispersed-reinforced designs on a base of cement-sandy concrete having a specific reinforcing surface $K \geqslant 2$ cm^2/cm^3.

3. In the reinforced concrete designs, there is permitted an additional reinforcement of the extended (elongated) zones by reinforcing rods.

4. The designs of slight thickness, reinforced by steel screens and reinforcement rods having $K < 2$ cm^2/cm^3, in the practice are also tentatively said to be of a reinforced-concrete nature. The instruction on the calculation of such designs is given in section 23 of the Present Rules.

II. MATERIALS

A. Concrete

5. For the production of the reinforced concrete designs, we should use the heavy cement-sandy concrete, brand 400 and higher.

Remark. The brand of concrete is adopted tentatively and is typified by the strength limit (kg/cm^2) for the compression of a concrete cube, with an edge of 7 cm, made from a concrete of working composition and tested after an aging of 28 days, in accordance with the standard ON9-373-62.

6. For the obtainment of the shipbuilding reinforcement concrete, we should apply Portland-cement, of a brand not lower than 500, of the following types: a) conventional; b) plasticized; and c) sulfate-resistant

The enumerated types of Portland cements should meet the requirements of GOST (State Standards) 970-41 and of the branch standard ON9-374-62 "concrete, shipbuilding, heavy. Materials for the production of concrete".

7. For the preparation of cement-sandy concrete, we should utilize the natural sands, meeting the requirements of the branch standard ON9-374-62 with the screening out of the grains coarser than 2,5 mm.

8. The water for preparing the concrete mixture should meet the requirements of the branch standard ON9-374-62.

B. Reinforcement

9. For the reinforcement of the reinforced-concrete designs, we utilize the steel screens and the steel low-carbon wire.

10. The webbed steel screens should meet the technical specifications imposed by GOST 3826-47.

11. The recommended numbers of screens (meshes) and their specifications are presented in Table 25. /177

Table 25

Characteristics of the Recommended Webbed Screens

No. of mesh screens according to GOST 3826-47	Diameter of wire, mm	Number of longitudinal & transverse rods per sq m of screen	Specific surface of reinforcement, K_n cm²/cm³, obtained at saturation of element with 1 cm thickness, by one mesh screen	Coefficient μ of reinforcement in one direction, obtained at saturation of element 1 cm thick by one mesh screen
5	0.7	350	0,770	0.00672
6	0,7	300	0,660	0.00573
7	0,7	260	0,572	0,00600
8	0,7	230	0,506	0,00441
9	1,0	200	0,628	0,00785
10	1,0	180	0,570	0,00715

11. The steel low-carbon wire utilized for the additional reinforcement of the extended zone of the reinforced concrete designs and for replacing parts of the webbed screen according to the engineering concepts (see Section 35, 36) should meet the requirements of GOST 3282 46.

III. STANDARD CHARACTERISTICS

12. The elastic-strength characteristics of reinforcing concrete on a base of concrete brand 400, reinforced by steel screens and having a specific reinforcement surface $K_n = 2$ cm^2/cm^3 are adopted according to Table 26.

Table 26

Standard Characteristics of Reinforcing Concrete

Type of stressed state	Standard resistance, kg/cm²	Elastic modulus, kg/cm²
Elongation.	$R_p = 65$	$E_p = 50\,000$
Compression.	$R_c = 320$	$E_c = 200\,000$
Elongation during bending. .	$R_{p.u.} = 120$	$E_{p.u.} = 50\,000$
Compression " "	$R_{c.u.} = 320$	$E_{c.u.} = 150\,000$
Cleaving	$R_{ck} = 65$	—
Shearing	$R_{cp}' = 100$	—

13. The magnitudes of the values of the standard resistances and the elasticity moduli of the cement-sand concrete, reinforced by a small number of webbed screens with $0.5 < K_n < 2.0$ cm^2/cm^3 (in this connection, in all cases the number of screens is not less than 2), are adopted in conformity with the instructions in Table 27. /178

14. The coefficient of the relative transverse deformation of reinforced concrete (the Poisson coefficient) is assumed to equal $\nu = 0.12$.

The shear modulus of reinforced concrete is determined from the formula $G = 0.45\ E_c$.

Table 27

Standard Characteristics of Cement-Sandy Concrete
Reinforced by a Few Webbed Screens

Type of stressed state	Standard resistance, kg/cm²	Elasticity modulus, kg/cm²
Elongation..................	$R_p = 20 K_n - 25$	$E_p = 50\,000$
Compression.................	$R_c = 320$	$E_c = 200\,000$
Elongation during bending...	$R_{p.n.} = 35 K_n + 40$	$E_{p.n} = 150\,000$
Compression " "	$R_{c.n} = 320$	$E_{c.n} = 50\,000$

IV. BASIC DESIGN CONCEPTS AND STRENGTH STANDARDS

15. The calculation of the design strength should indicate that for it, there is assured the required strength reserve, i.e. under the effect of the external design forces, the stresses in the design do not exceed the tolerable values.

16. The volume of the strength calculations, represented in the engineering plan is established by the Registry of the USSR, by the River Registry of the RSFSR, or by the buyer in dependence on the type and purpose of ship. In a general case, there should be presented:

a) A composite table of the weights of the hull and of the variable cargos, distributed through 20 theoretical compartments, with a brief explanatory listing of the procedure used in compiling this table;

b) An instruction on the loading of the ship;

c) The calculations on determining the bending moments and the transverse forces;

d) Calculation of general strength;

e) Calculations of local strength; and

f) Detailed instructions on the modified standards or on the specification of the standards not envisaged by the existing Rules.

17. The calculation of strength, in a general case, is subdivided into the following individual parts:

a) A determination of the value and nature of the design loads;

b) A determination of the maximum stresses in the design sections for the adopted design loads; and

c) The specification of the values of the hazardous stresses, the establishment of the standards of tolerable stresses and a verification of the strength.

18. The calculations should be accessible for an exhaustive verification of all of the data included in them based on the design materials and based on the references to the sources.

19. The external design loads, acting on the ship hull and on its individual parts are established in accordance with the instructions of the effective standard documents of the USSR Registry, or of the RSFSR River Registry, with allowance for the requirements of the technical project for the planning and section 20 of the present Rules.

20. The actual hull weight is determined on the basis of the measurements, adopted in the plan, for the members of its designs. /179

The volumetric weight of the reinforced concrete is found from the formula

$$\gamma_{жб} = \gamma_{пб} + 11\mu,$$

where μ = the reinforcement factor of the cement-sandy concrete in one direction; and $\gamma_{n\delta}$ = the volumetric weight of the cement-sandy concrete determined experimentally in each instance. In the absence of experimental data, in the preliminary calculations, we have decided to assume $\gamma_{n\delta}$ = 2.20 tons-m^3.

21. The values of the stresses developing in the reinforced concrete design during the effect of the design load on it, are established according to the general rules of the structural ship mechanics under the assumption that all the ship material is isotropic and under the effect of the design loads functions as elastic material. In this context, the calculation of the designs for bending is

conducted with allowance for the difference in the values of the elastic characteristics for the compressed and extended (elongated) zones.

22. The reinforced concrete designs, reinforced additionally by rods, are calculated according to the elastic stage of the functioning of the reinforced concrete according to the instructions of the existing Rules, with allowance for the reduced section of the element (member). The reduced section of the member is determined proceeding from the relationship of the standard resistance of reinforced concrete and the calculated (designed) resistance of the rod-type reinforcement; in this connection, the design resistances of the reinforcement rod for all types of steel are assumed to be:

for the reinforcement of the extended elements R_a, kg/cm^2....1200

for the reinforcement situated in the extended zone of the bending elements R_a, kg/cm^2..............................2400

The rod-type reinforcement located in the compressed zone of the section of reinforced concrete design is not considered in the calculation.

23. The calculation of the sections reinforced by a small number of steel screens with $0.5 < K_\eta < 2.0$ cm^2/cm^3, is conducted in the same way as the calculation of the reinforced concrete sections. In this connection, the values of the standard resistances R_i are determined on the basis of Table 27, while the design resistances of the rod-type reinforcement for all types of steel are determined from Table 28 depending on the value of the coefficient of specific reinforcement surface, K_η.

For the intermediate values of K_η, the values of the design resistances of the rod-type reinforcement are determined by linear interpolation.

24. For the dangerous (standard) stresses during elongation and elongation during bending, we adopt the stresses corresponding to the moment of opening of cracks by the amount of 0.01 mm, at a thickness of the protective layer of 2 mm; /180

during the compression and compression under bending, we adopt the stresses equalling the prismatic strength of the cement-sandy concrete.

Table 28

Design Resistances of Rod-Type Reinforcement Utilised for Reinforcing the Reinforced Concrete Designs

Specific surface of reinforcement, K_{n} cm^2/cm^3	Design resistance, kg/cm^2	
	reinforcements of elongated elements	reinforcements located in elongated zone of bending elements
2.0	1200	2400
1.5	950	1900
1.0	700	1400
0.5	450	900

The standard resistances of the material on a base of cement-sandy concrete, reinforced by webbed steel screens, are determined according to the instructions in section III.

25. The permissible stresses are specified as a certain fraction of the standard resistances according to the instructions in Table 29.

Table 29

Standards of Permissible Stresses

Design loads	Attainment of critical value by standard stresses		Attainment of critical value of main elongating stresses
	elements participating in overall strength or in overall & local strength jointly, & also the compressed members	elements taking part only in local strength (other than the compressed elements)	
Constant..........	0.65	0.75	0.55
Constant & random, & random only. . .	0.75	0.80	0.65
Emergency.	0.85	0.90	0.70

The strength of the design is considered to be guaranteed if the total effective stresses do not surpass the values of the permissible (tolerable) stresses.

In this connection, if by the calculation for the simultaneous effect of the constant and random forces, or by the calculation for the emergency loads, there is provided the required strength reserve (see Table 29), it is necessary that under the effect of only the constant forces, there would be assured the strength reserves not lower than those indicated in this table.

26. In the necessary cases, in addition to verifying the strength of the design in respect to stresses, we should verify the stability both of the entire design as a whole, as well of its individual members, and also we should verify the maximum stresses.

27. The compressed elements during the flexibility $l_o/r < 50$ for stability were not verified. In the calculation of the compressed elements for stability at the flexibility $l_o/r > 50$ (for the rectangular section $l_o/b > 14$), the permissible stress to compression is decreased by multiplying it times the coefficient of longitudinal bending,

l_o/r	l_o/b	φ	l_o/r	l_o/b	φ
50	14	1	104	30	0,50
55,4	16	0,88	110	32	0,46
62,2	18	0,80	117	34	0,44
69	20	0,73	124	36	0,40
76	22	0,67	131	38	0,37
83	24	0,62	137	40	0,35
90	26	0,57	144	42	0,32
97	28	0,53	150	44	0,30

Remark. At the intermediate values l_o/r or l_o/b, the φ-values are adopted by linear interpolation. /181

The design length of the element l_o is obtained by the multiplication of its actual length times the coefficient, depending on the method of reinforcing (attaching) the ends. The coefficient has the following values:

during the rigid fastening of the ends..................0.5

during the hinged fastening of both ends................1.0

in case of one rigidly fastened and one flexibly-fastened end..................................0.7

in case of one rigidly fastened and the other freely-mounted end...............................2.0

in case of the partial fastening of the ends and in the frames with non-displacing units.......0.7

in the frames with displacing units................ 1.0-1.5

28. The verification of the maximum stresses of the bending elements should be conducted on the basis of the formulas from the ship structural mechanics, proceeding from the actual geometric dimensions of the design and of the reduced elasticity modulus E_{np} for sagging, or from the reduced stiffness of the element, taking into account the difference in the elasticity moduli of the elongated and compressed zones and the inertial moments of these zones.

The values of the reduced elasticity for bending, E_{np} for the loads, not exceeding the maximal operational loads under the conditions:

brief (transient) effect of load E_{np}, kg/cm^2........200,000

prolonged effect of load, E_{np}, kg/cm^2................100,000

The maximal sagging must not exceed 1/300 of the value for the designed span.

29. The combined designs, representing a combination of the beams made of ordinary ferroconcrete and the plates of slight thickness, reinforced by the webbed steel screens, and also with screens and individual rods, should be calculated on the basis of the principal of the method for the breakdown loads.

30. In a determination of the value of the breaking load in dependence on what part of the section (compressed or elongated) the plate is located, we should distinguish two cases:

a) If the plate is located in the section's compressed zone, the breaking force is determined as for the tee-type beams made of ordinary ferroconcrete, without allowance for the webbed screens of the plate;

b) If the plate is located in the elongated zone of the section, the breaking force is established proceeding from the reduced area of the plate and the value of the standard resistance of the material, guided by the instructions

in sections 12, 13, 22, and 23 of the present Rules. In this context, we overlook the functioning of the concrete in the beam to elongation, while the elongated reinforcement of the beam is introduced into the calculation with the stress equalling the design resistance R_a of the reinforcement.

31. The design width of the plate, functioning together with the beam, is assumed to equal the least of the following values: one third of the design span of the beam; the half-sum of the plate's spans adjoining the beam; or 25 times the thickness of the plate.

32. The strength of the combined designs is regarded as assured, if the coefficients of strength reserve obtained as the ratio of the design breaking force to the design force from the effective load, do not surpass the corresponding values, governed by the existing Rules for the construction of ferro-concrete ships, while the crack-resistance of the beams in the elongated zone of the section complies with the requirements of these same Rules. /182

V. INSTRUCTIONS FOR DESIGNING

33. The thickness of the designs made of cement-sandy concrete reinforced by steel screens is adopted according to calculation, but is not less than 10 mm.

34. The thin-walled designs made of cement-sandy concrete should be reinforced by the steel gauze screens in order that the specific surface of the reinforcement would fall within the limits of $2.0 \leqslant K_\eta \leqslant 3.0$ 1/cm.

35. The steel screens should be distributed evenly through the height of the design section.

According to the engineering concepts, it is permitted, in the central third of the section height of the bending elements, to conduct the replacement of the fiber (mesh) screens by rods, and to leave the standard specifications of the material without change.

36. The rod-type reinforcement introduced into the calculation should be situated between the mesh screens; moreover, from the side of the protective layer of the elongated zone there should be not less than two mesh screens.

The diameter of the reinforcing wire is established depending on the element's thickness, and should not exceed 5 mm.

37. The thickness of the protective layer in the designs under all forms of reinforcement, independently of the size of the specific reinforcement surface, must equal 2 mm.

38. The joining of the mesh screens in the elements should be conducted with lap joints, with an overlapping of the ends by not less than 10 cm. The joints of one layer of the screen should be displaced relative to the joints of the other layers in order that in any section of the element, there would not be more than one joint.

39. The projections of the rod reinforcement utilized for connecting the beams with the plates and for attaching the inserted parts, should be introduced between the mesh screens of the plates for a length of not less than 40 diameters.

40. The joints of the sectional (prefabricated) members are envisaged by the plan. The design of the joints should not disrupt the strength or the water-tightness of the design as a unit. In the connection of the reinforced-concrete plates, it is necessary to provide for the bypassing (overlapping) of the mesh screens into the zone of the joint.

41. For purposes of providing the corrosion resistance of the designs made of cement-sandy concrete, reinforced by steel mesh screens with a protective layer of concrete equalling 2 mm, in the required cases, one should envisage: the application of protective coatings to the surface of the design; the application of mesh screens with an anticorrosive coating; and the introduction of inhibitors into the concrete.

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